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Railroad Electrification By far the most interesting news item in the railroad field for some time was the announcement made at the meeting of the Canadian Society of Civil Engineers last week that the Pennsylvania had adopted alternating current for its suburban electrification at Philadelphia and that the Chicago, Milwaukee & Puget Sound would use 2400-volt direct current for its 440-mile electrification in Montana. No details as to the reasons for the selections were announced and, consequently, there can be only speculation as to the factors which influenced the conclusions. If the grounds were entirely technical and based solely on the immediate problems in hand, the decisions are decidedly more interesting than if other considerations had some weight, for both decisions seem opposed to the evident tendencies in electrification. The tendency for suburban and terminal work certainly is in the direction of direct current and for trunk-line electrification toward alternating current. There is a possibility that the Pennsylvania's decision was influenced by the possibility of trunk-line electrification in the neighborhood of Philadelphia. It will be noticed, however, that this speculation regarding the Pennsylvania's reasons goes exactly contrary to the Puget Sound's decision. For that very reason the figures would be most interesting. Until that railroad speaks one can only speculate whether some factor outside of technical merit had any considerable weight in reaching the decision.

Pay of Technical Officials The compensation of technical men in the Government service and its effect upon the stability of the personnel are brought up in the annual report of the director of the U. S. Geological Survey. He points out that in the four and one-half years ended January, 1913, the number of geologists who left the Government service for the primary purpose of bettering their financial condition was 41, and that these men had received salaries outside the public service amounting to an average immediate advance of 149 per cent; in other words, their new salaries were two and one-half times those paid them by the Geological Survey. Consequently there is more or less instability in the service, which is especially unpropitious for the execution of the best work because the positions vacated are generally the most important. The only optimistic feature of the situation, the director feels, is that the loss to the Geological Survey can be regarded as one measure of the educational contribution that the public service is making to the world's work. The condition described is not peculiar to this branch of the Government service. The Office of Public Roads, for example, is constantly losing valuable men, but there, again, a philosophical view is taken and the loss is counted a gain to the counties and States to which these men have carried the high standards of the Government's highway branch. The demand for Government men is not hard to understand. They have more time for research and careful work than those engaged in private business. Hence, if they have aptitude they are developed rapidly and receive a recognition, which, while fully deserved, comes in part, too, from the fact that they are Government officials. Whether some scheme can be devised to hold these men in Government service against the lure of gold is questionable. If it is ever done it will be by making Government service so desirable as to counterbalance the financial attraction in private work. In the army and the navy this end is accomplished both by the standing of those organizations and the assured pension at the end of the service. A recognition equal to that enjoyed by the army or navy is hardly to be expected for many years, if at all. Nevertheless, recognition of a high order is accorded all public officials abroad, while pensions are inseparable from Government service. The alternative is the payment of salaries equal to those in private work, but this, Government officials seem to feel, is the most remote of the possible remedies.

Designing for the Future To point out the danger of determining types of structures or processes years before they are built or used is to assume a bad condition which is not common. But not infrequently portions of work—foundations, for example—are put in when the primary installation is made, leaving the completion of the work until such time as increased capacity is required. In some lines advancement is so rapid that installations only a few years removed from each other in date of construction are radically different in type. The later installation is naturally the more efficient, and when the increase in efficiency is very marked it is sometimes advisable to scrap the earlier equipment, no matter what its remaining life. While these are elementary considerations, recent occurrences give them point. In editorials in this journal on Oct. 25 and Nov. 22 attention was called to the fact that advances in hydroelectric practice had been so rapid that even before the great Keokuk plant was put in operation certain features of the equipment had become obsolete. More recently it has been rumored that in a large installation where foundations were prepared for double-runner wheels, single wheels will be used instead. The moral is that in building any portion of a future structure, whether in hydraulic or other branches of engineering, the work should not be carried beyond the simplest stages. Just how far it is advisable to go must depend entirely upon the judgment of the engineer in charge and his knowledge of the history of the art. In some lines there is a fair degree of certainty that the general types now in use will be adhered to for some time, while in other branches practice is in such a state of flux that there is possibility that even the most simple foundation work will need to be destroyed when additions are necessary. This factor has risen in connection with the design of the terminals for the New York State Barge Canal. It was concluded that nothing but the docks should be constructed at the present time, leaving the design of superstructure for a later date, so that it may be made to conform to whatever type of handling device is proved to be most economical by the time the terminals are to be put in operation. Again, in the papers on the Imhoff tank published in this journal recently the same idea was emphasized. The advances in sewage disposal are so rapid that the construction of tanks which are not needed for four years or so is hazardous. The point is well worth bearing in mind when conditions dictate the advisability of putting in some portions of a structure not needed at the present time. There is danger of forcing the use of equipment or processes which will be obsolete a few years hence, or of losing large sums in discarding or removing the partly constructed work to make way for the more efficient methods.

Railway Dissolution Rumor has it that the New York Central lines are to be the object of the next Federal dissolution suit under the Sherman law. The Engineering Record makes no attempt to judge the applicability of this much-discussed law to this case. It is pertinent, however, to look a little into the history of the New York Central system and to speculate as to the probable results of unscrambling it. The New York Central & Hudson River Railroad proper was itself made up more than half a century ago of fragmentary, independent lines which had to be knit together to become either profitable to their owners or satisfactory to the public. The amalgamated lines prospered, paying 8 per cent dividends. Then the West Shore Railroad was built. Paralleling the New York Central closely but missing several of the principal cities, it never paid, but its competition cut away about a quarter of the New York Central's profits. The older road was thus forced in self-defense to buy the new. West of Buffalo the Lake Shore and the "Nickel Plate" present a similar situation. Like the West Shore, the "Nickel Plate" was built to sell, as those who know how the latter road acquired its nickname can testify. It is yet to be demonstrated that two inferior railroads side by side throughout their length serve the public better than one good one. Now, consider the effect of divorcing these lines. The "Nickel Plate" still retains its identity, although the recent efforts of New York Central interests point to its more complete absorption. The West Shore, however, has long since ceased to exist as a separate operating entity. Up the Hudson it serves its own population, and is bound to do so. Up the Mohawk a few fast trains run, although the faster they are the sooner they cross over to the superior roadbed of the older line; there is also some local business, but this section is chiefly useful as fifth and sixth tracks for the main line. The section from Utica to Syracuse is electrified and operated by a separate company. From Syracuse west the line has during the last decade undergone extensive grade revisions to fit it for use as an adjunct to the main line. As such it has become very valuable to the New York Central. As an independent railroad it would sell for little, and would probably be as little able to earn a living as it was originally. A few years ago, when permission was sought to build a third line from Buffalo to the upper Hudson, it was shown to the satisfaction of the Public Service Commission that the existing lines served the territory adequately and that there was not enough business to support another road. The New York Central situation probably has many parallels. The difference should be recognized between frenzied buying up of all remote competitors at their own prices to effect a general monopoly and the forced protection of a company's legitimate business from great inroads by irresponsible promoters.

Battle of the Currents

For several years little has been heard of the battle of the currents which began after the original electrification of the New York Central and the New Haven at New York had been completed. Echoes of the controversy were heard at Montreal on Dec. 18, when Mr. A. H. Armstrong, of the staff of the General Electric Company, read a paper on electrification before the Canadian Society of Civil Engineers. An abstract of the address will be found on page 716.

The Engineering Record has no intention of reopening in its columns the controversy, but believes it desirable to point out that alternating current for heavy steam-railroad electrification is not to be dismissed as lightly as it was by Mr. Armstrong. His paper contains a list of "Main-Line Electrifications" in the United States and Canada, but of the installations listed only those of the New Haven, the New York Central and the Norfolk & Western can truly be classed as heavy trunk-line electrifications. The others are more nearly of the terminal type. For example, there are listed as main-line electrifications the St. Clair, the Detroit River, Hoosac, the Cascade and the Baltimore & Ohio tunnels, and also the terminal electrification of the Pennsylvania Railroad at New York. The installations on the Butte, Anaconda & Pacific and the Canadian Pacific, also listed, are, on account of their short length and small number of locomotives, in the terminal rather than the main-line class. In other words, when analyzed it appears that the table, though entitled "Main-Line Electrifications," contains only three roads which should be so classed, provided this term is interpreted as meaning "trunk-line electrification"—the construction a steam railroad man would put on it. As a consequence any conclusions based on the table will necessarily be erroneous. Had it been limited to the strictly trunk-line electrifications mentioned above there would likely have been greater hesitation in dismissing the alternating current.

Even if one were to go no farther than the American experience exemplified by the New Haven, one might well hesitate before making a sweeping condemnation of the alternating-current system. True it is that the first few years of that installation were filled with stress and difficulty, but he would be a rash man indeed who would sweepingly say at the present minute that it is not a success. The handling of 65,000 cars per month, a monthly locomotive mileage averaging 145,000 and speeds for the heaviest passenger trains of 65 and 70 miles an hour do seem to indicate that the alternating-current system deserves more consideration than was given to it in the paper presented at Montreal. Within a short time, moreover, the New Haven trains will run through to the city of New Haven, 75 miles from New York, on electric power. Not only will the passenger service be handled electrically, but the freight trains will also be moved by that means.

One need not be content merely with American experience. Alternating current has demonstrated its suitability for trunk-line work in Europe, and in lieu of fuller experience here it is logical to turn to foreign practice. Not only is sentiment on the Continent in favor of alternating current for heavy main-line service, but the experience in alternating current of the London, Brighton & South Coast Railway in England has been very satisfactory.

For interurban and terminal work there is no doubt that in general direct current is the more suitable. But a clear and distinct line should be drawn in the minds of engineers between interurban work and trunk-line electrification. It should not be difficult to see the difference between the running of two or three interurban cars in a 10-mile stretch and the same number of heavy passenger trains making 60 to 70 miles per hour. The power demands for the latter condition are so much greater than for the former that the transmission problem is in an entirely different class. Any consideration of the electrification problem which fails to take in the locomotive and the transmission system is certain to lead to unwarranted conclusions.

Fortunately, the paper was well answered at Montreal, and it is not unlikely that we shall not again witness in a technical society so offhand a dismissal of the alternating-current system, unless, of course, departures in the relative merits of the two undergo a radical change and to the advantage of the direct-current system.

Government Ownership Again

Government ownership of railways was commented upon in this journal last week, but before the issue reached its readers the newspapers of the country were not only discussing the postmaster-general's report advocating Federal ownership of the telephone and telegraph lines, but also the dissolution of the Bell Telephone-Western Union merger and the report of the Interstate Commerce Commission on the parcel-post and railroad control. In this dissolution matter the Government secured all it asked for—separation of the interests and the admission of the toll messages of the independent companies to the Bell wires. This agreement, which received the hearty commendation of the President, should have a tendency to check the Government-ownership talk, though that talk is not likely to be long stilled in view of the present evident drift with reference to public utilities.

The tendency concerning these utilities was plainly evidenced in the Interstate Commerce Commission's report. Extension of its control over the construction, maintenance and operation of the physical properties of the railroads was recommended, and the former recommendation that it be given supervision of stock and bond issues was reiterated. The first of these recommendations was made in order that greater safety might be promoted. Whatever the reason, however, the effect is evident—to remove from the railroads control over matters heretofore left entirely to their management. Certainly the safety of passengers is a concern of the Government, but whether the type of supervision suggested by the commission is the best means for insuring it is a moot question. Regulation by the commission has been accepted by railroad officials as necessary and even desirable; the debatable question is the extent to which it should be carried.

A surprising feature of the commission's report is its request that it be relieved of the responsibility of passing upon parcel-post rates. Its reason is that it must regulate the rates of the express companies, and at the same time endorse parcel-post rates without regard to their effect upon those companies. The commission indicates that the reduction of parcel-post rates might "threaten the destruction of the existing express companies before adequate provision has been made by the Government for taking over all such business. In such a case the commission would be in grave doubt as to its duties under the present law." It is evident that the task is not an easy one for the commission, but, nevertheless, it is clearly its duty to retain the responsibility, for only then is there likelihood that the effect of the expansion of the parcel-post system on the express companies will be clearly appreciated by any Government agency. There was justification for the establishment of the parcel post, even though the business of the express companies was certain to be affected. They have seen the movement coming for years, and were, or should have been, prepared. At the same time there is no logic in separating the function of regulating the rates of the two institutions, in order that one body may proceed on a destructive course without cognizance of what it is doing.

It is apparent, therefore, that the comments on Government ownership in this journal last week were well timed. A few days' delay would have furnished additional text for the discourse.

Hetch Hetchy Passes into Political History

Politically Hetch Hetchy has passed into history, mourned by the enthusiasts who let their ardor for scenery run away with their judgment. The President signed the Raker Bill, giving San Francisco the rights it sought, on Dec. 19. In approving the measure the President attached a memorandum which puts the situation succinctly.

"I have signed it," he says, "because it seemed to serve the pressing public needs of the region concerned better than they could be served in any other way, and yet did not impair the usefulness or materially detract from the beauty of the public domain.

"The bill was opposed by so many public-spirited men, thoughtful of the interests of the people and of fine conscience in every matter of public concern, that I have naturally sought to scrutinize it very closely. I take the liberty of thinking that their fears and objections were not well founded.

"I believe the bill to be, on the whole, in the public interest, and I am the less uncertain in that judgment because I find it concurred in by men whose best energies have been devoted to conservation and the safeguarding of the people's interests, and many of whom have, besides, had a long experience in the public service which has made them circumspect in forming an opinion upon such matters."

Commenting on the President's action, the New York "Evening Sun" says very sensibly: "Extremists of landscape conservation will continue to cry that natural beauty is above any price, forgetting that the same is true of human life, which most of us retail in lengths of week, month or year for the price of pay; true also of the right of a great community to live, grow and follow its destiny. The real trouble is in finding the true ratio between great values, all of them beyond mere money worth."

Thus is closed, so far as the National aspects are concerned, a controversy of twelve years' standing, beginning with the recommendation of the city engineer in 1901 that the Hetch-Hetchy and Lake Eleanor sites be filed on. Whether from the technical aspect Hetch Hetchy is a dead issue remains to be seen, for though the city's engineers state that they are satisfied with the measure passed there are those who believe that the concessions to the Modesto-Turlock interests were far too liberal and that the water actually stored will be a disappointment. There is a possibility, therefore, that the valley with the funny name may transfer its battles from the arena of National politics to that of the technical societies.

Unification of Weights and Measures

Some recent reports as to the increasing use of the metric system indicate that material progress is being made toward a really international system of weights and measures. Of course for scientific purposes the metric system has held this position for a good many years, but it is one thing to secure such a measure of uniformity and quite another to overcome the inertia of the man in the street far enough to make him think in new terms, whether of metrology or of anything else.

The most notable item in the recent history of the subject is the beginning of a change from the old standards in Russia, which has now officially adopted the metric system for some purposes and has put itself on record as favoring its general introduction. This step completes the agreement of all Continental Europe in a single system, although the final working out of the details into universal use necessarily will take much time. Japan is working officially to the same end, the metric system being taught in all the public schools there. China under its new government has likewise taken preliminary steps in the same direction, and even Siam has introduced the metric system in its public works, and, having joined the International Convention, is setting up a central bureau of standards at Bangkok. Only the English-speaking countries are conspicuous in sticking officially to their ancient system, and the necessities of world trade are pressing heavily to force them into line.

All academic questions aside, the unification of the world's weights and measures is an end greatly to be desired and inevitably coming. It is perhaps not to be wondered at that progress has been slow, for probably nothing save language is more persistent than national systems of weights and measures. The necessities of international intercourse steadily force unity in such things, and it is very noteworthy how even in the same country a tangled system of obsolescent weights and measures persists in the backwoods regions long after its inconvenience has enforced simplification wherever people deal with each other often and on a large scale. For example, of English measures of length the furlong is nearly obsolete and the rod obsolescent, clinging most persistently in the extreme country districts. Of smaller English units to be found in the old arithmetics, the barley corn has dropped out of sight entirely, and the nail persists only locally, even in England. Other units once common have vanished except for special purposes. For instance, the hand is only used for defining the height of horses, and the avoirdupois dram survives only in defining the powder charge for shot-gun cartridges. The same sort of survivals turn up in other countries as well, like the use of the Paris line by the French opticians, the corresponding inch having almost entirely disappeared, like the long German inch. All over Europe there are similar curious instances interesting now only from a psychological standpoint. One of the most persistent, the old carat for weighing diamonds, has recently, by international convention in the trade, finally been put out of business by the adoption of the metric carat of 200 milligrams.

This change leads to a curious reflection on the psychology of the matter, which perhaps may explain part of the obstinate and apparently unreasoning objections to other metric units. The metric carat is slightly smaller than the average of the carats previously in use, so that the weight of a stone in the new system is expressed by a number a little larger than in the old. It is interesting to speculate on the probable attitude of some of the most uncompromisingly hostile industries were the meter a little shorter than the yard instead of a little longer, and the liter a little less than the ordinary quart, instead of somewhat greater.

A general agreement on such matters would undoubtedly be more easily reached were the arguments pro and con less lacking sometimes in frank appreciation of real conditions. It is certainly no valid argument against the metric system to assert that a change from English to metric measures is rendered enormously difficult by the numerical relation between inch and centimetre, for works like several in England and many on the Continent in which the change has actually been made recently have found little difficulty; nor, on the other hand, is it a strong argument for the metric system to make caustic criticisms on the number of kinds of bushels which have been legalized in various States. These mark, in fact, merely a transition from bulk measurement to weight measurement, in which an average uniform bulk of various commodities is denoted in terms of the weight which should fairly yield that bulk. At all events there can be no doubt of the desirability of a uniform international system for ordinary weights and measures, just as we have an international system of time keeping, and local prejudices in the one matter are just as certain to disappear as they are in the other.

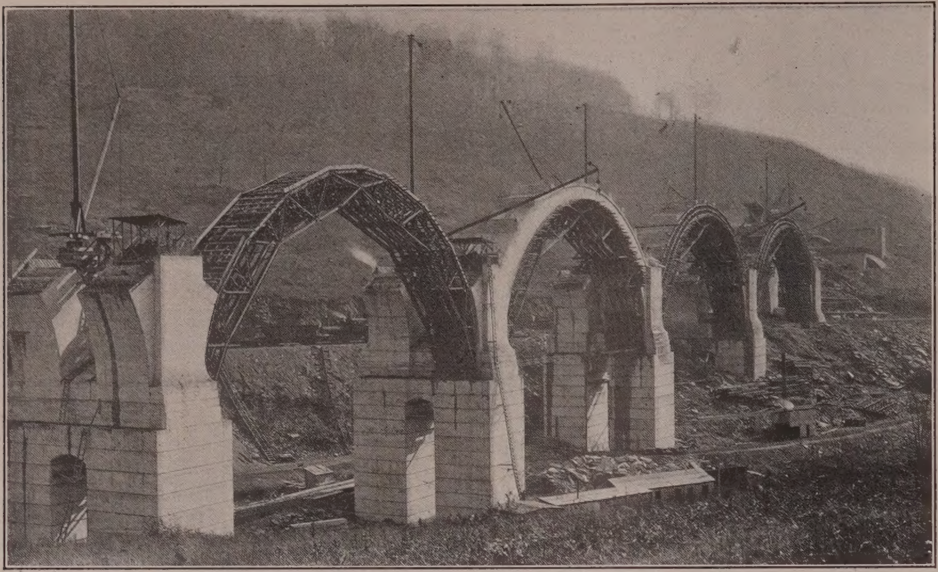
The Engineer and the Accountant

In the administration of large manufacturing plants and public-service corporations the combined abilities of specialists in engineering, law, production, salesmanship and accounting are essential to an efficient conduct of affairs. Between the first and the last-named class of experts a wide gap formerly existed, but the broadening tasks of each have brought the engineer and the accountant into closer relations, notably in connection with the work of commission regulation, the study of operating economies and the more extensive tabulation of construction costs. Around the conference table of the efficiency committee the accountant and the engineer come into especially effective contact.

A few specific illustrations of the sort of co-operation that is needful may be given. In practically all cases where a company appears before a State board seeking permission to issue securities it is required to present a list of the expenditures which it is proposed to capitalize, and usually at the stock or bond hearing following the petition a long exposition of these expenditures occurs. Often the lists are inadequate as to detail, and insufficient data are given to enable the commission to compare the actual or anticipated costs with similar expenditures elsewhere. It is most important that all legitimate costs shall be included in such exhibits, and here the accountant's broad knowledge of classifications and the segregation of items, combined with the engineer's understanding of what is essential in the way of physical data, may be of the utmost value. Unless an engineer has had a good deal of experience in court and commission work he is liable to overlook features of expense which the accounting expert will set forth at the outset. On the other hand, if the data are left for the accountant to compile he is almost sure to overlook some fundamental engineering dimension or quantity, without which no intelligent opinion can be rendered as to the propriety of the expenditure. In an extreme case it may lead to a scaling down of the amount permitted to be capitalized, which means that the burden must be borne out of immediate earnings by reduction of a hard-won surplus, or by slicing the depreciation allowance. The only safe way is for both engineer and accountant to go over exhibits of this character with the greatest care before delivering them to the executive or legal advisers conducting the company's case.

In valuation work the accountant needs the assistance of the engineer in the interpretation of items of plant expenditure, in the analysis of construction costs, and in the treatment of depreciation. The engineer's opinion as to the life of equipment and its fitness for service after a stated period must largely guide the accountant in his consideration of replacement costs, and the former obviously is the best interpreter in an operating organization of maintenance expenses. No accountant without engineering training or advice is in a position to judge whether repair costs are normal or abnormal in relation to the service conditions. The engineer, for his part, needs the comprehensive viewpoint of the student of figures in making sure that his subordinates keep adequate records and properly classify accounts so that unusual conditions can be noted with the least possible delay. In the working out of cost systems these two classes of experts should confer at sufficient length to yield results of interest to the technical man no less than to the bookkeeper. Engineering advice may be as useful in the accounting department as in the purchasing bureau, and the recommendations of the accountant ought to make the work of the engineer easier if the classification adopted in a given company is the joint product of their labor.

It is surprising how difficult it is to maintain an accounting system which will give at short notice the variety of engineering or operating data which may be demanded in a special investigation, such as an analysis of wages and hours of service on a railroad; but this is a result of the expense of multiplying statistics which are really by-products of of a company's bookkeeping records. There seems to be no limit to the amount of analytical study that can be applied to a payroll of this kind. In the arbitration of the hours and wages of street-railway employees now drawing to a conclusion at Boston one of the most remarkable features of the inquiry has been the amount of extra work required from the auditing department of the Boston Elevated Company in the preparation of special information from its routine records to enable the cost and duration of platform labor to be thoroughly analyzed. The company's accounting system is exhaustive and scientific; but it is obvious that all the combinations of data which may be of possible engineering or operating use after being derived from such records cannot as a practical matter be maintained from month to month without undue expense. On such a system close relations between its engineering and accounting staff constantly tend toward the production of records of real value in the solution of special problems and toward the economy of time in studying the operation of the road from special angles of inquiry. In defining just what sort of statistical data are desirable for executive deductions there is a field for larger mutual service for the engineer and the accountant than has often been developed in the past.



First Position of Arch Rib Centers and First Ring Constructed

Construction of the Martin's Creek Viaduct

Three-Track Structure 1600 Feet Long and 150 Feet High with 150-Foot Concrete Arch Spans Built on Steel Truss Centers

NEXT to the Tunkhannock viaduct the Martin's Creek viaduct is the most important structure in the 40-mile relocation of the main line tracks of the Delaware, Lackawanna & Western Railroad between Clark Summit and Halstead, Pa. It is built entirely of concrete, with long and lofty spans on deep foundations carried to solid rock through beds of clay and boulders. It is of massive construction and conservative design, adopted after consideration of other types to secure maximum safety, durability and economy. The methods of construction, the difficult foundations and the equipment for handling large quantities of concrete and for building the heavy arch spans at a great height are the principal features of this structure.

GENERAL DESCRIPTION

The viaduct is a three-track structure 1611 ft. 8 in. long and 150 ft. high above the creek. It carries the track about 90 ft. above the former elevation and crosses over the old main line, thus helping to provide a new alignment that will reduce grades and curvature sufficiently to justify the $12,000,000 expenditure for the cut-off improvement. The viaduct will be 48 ft. 4 in. wide over all at the arch rings and will have two full-centered spans of 50 ft., two of 100 ft., and seven three-centered spans of 150 ft. with a rise of 59 ft.

Each span will have two arch ribs 12 ft. apart in the clear. For the 150-ft. spans these ribs will have a width of 17½ ft. and a thickness at the crown of 6 ft. and will each contain 1000 yd. of 1:3:5 concrete. They will carry solid transverse spandrel walls 12½ ft. apart supporting floor arches and parapet walls. The west end span is made of two-arch ring segments of 100-ft. span, forming an abutment. The concrete floor system is carried to the center or crown of the abutment arch, with the adjacent fill extending to the end of the floor; the toe of the slope extends to the center of the westerly 150-ft. span, the 100-ft. arch being entirely covered by the embankment. The east half of the westerly 100-ft. span carrying the bridge floor is inclosed on both sides, with spandrel walls reaching to the surface of the slope and giving the appearance of a solid masonry abutment. At the land end of this span the tracks are laid on the fill.

The completed structure will contain about 84,000 yd. of concrete and will involve about 25,000 yd. of foundation excavation. Except for its smaller dimensions, the viaduct design and construction are substantially the same as those of the Tunkhannock viaduct, described in the Engineering Record of May 3, page 484, and Nov. 29, page 594.

DELIVERY OF MATERIALS

Work was commenced in June, 1912, and a side track for material was connected with the main line. An additional track was also built parallel to the side track partly supported on a wooden trestle to provide for receiving, unloading and storing cars. From 400 to 500 cars of material are received here monthly, and about 8000 yd. of broken stone, 4000 yd. of sand and 4000 bbl. of Dexter cement are kept in storage. Sand and stone are dumped into heaps on the sidehill and are thence delivered by a Mead-Morrison clamshell bucket to storage hoppers at the concrete-mixing plant. At the mixing plant a 2-yd. Carlin cubical mixer is mounted about 10 ft. above the track. From it concrete is discharged into buckets and delivered over the service tracks to the work.

Concrete is distributed to the pier sites by a 3-ft. gage surface track which runs the full length of the viaduct and serves the power plant, shops and storage yards. There are in all eight switches, all of which are visible at a central point, where they are operated from an interlocking stand by one man, thus effecting a considerable saving of time and increasing the safety.

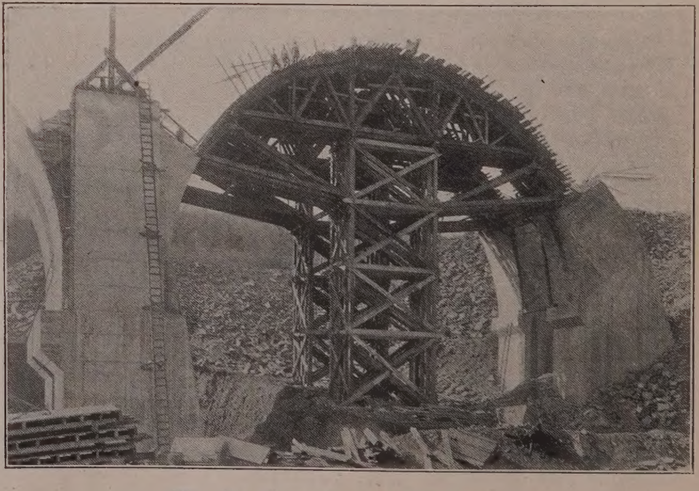
SAWMILL

An important part of the plant is the sawmill equipped with a ripsaw, bandsaw, cutoff saw, planing mill, and molding machine—all operated by one 25-hp and one 15-hp engine. Besides cutting and planing timber and doing miscellaneous work, the sawmill is of great importance in sawing the heavy timber into boards. For this contract a large amount of 12 x 12-in. yellow-pine timber was ordered, and most of it was first used for the construction of towers, trestles and for the heavy bracing in the deep foundation pits. As fast as released the timber is sawed up into smaller sizes and boards and is planed for use in the construction of concrete forms.

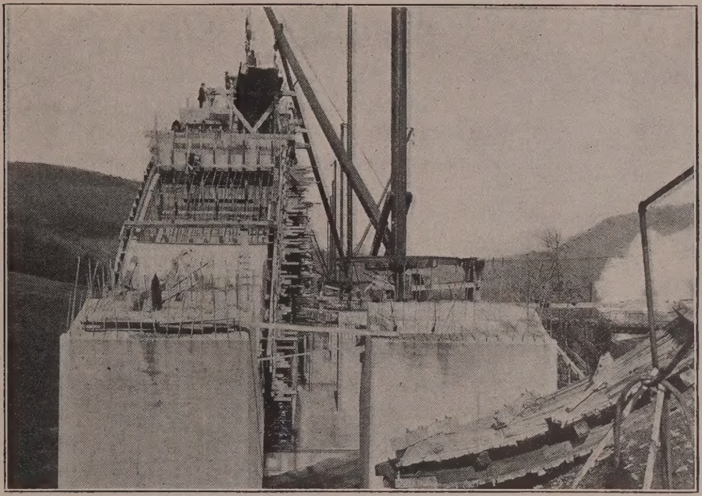
PIER FOUNDATIONS

About 30,000 yd. of earth were excavated at and near the pier site by a Marion steam shovel, which made a cut with an average depth of about 20 ft. parallel to the axis of the viaduct. The spoil was used to form embankments for the service tracks and to make a fill, on which the machine shop and concrete plants were located. The soil consists chiefly of compact water-bearing sand containing many small boulders and considerable gravel and overlies hard blue flagstone at a depth of from 18 to 70 ft. below the surface.

Completing Piers for Arch Centers



Wooden Centering for End Span



Concreting Voussoirs in One Arch Span

Span Concreted on Steel Centers

The pier foundations were excavated to sound rock in open cofferdams made by driving one or two tiers of steel sheet piling with a No. 2 Vulcan steam hammer operated in 40-ft. leads suspended from a derrick boom. Where two tiers of sheeting were required the inner tier was driven first and the outer tier was driven afterward at a 5-ft. distance in order to allow clearance for excavating the earth between them, as was explained in the description of the foundation of the Tunkhannock viaduct.

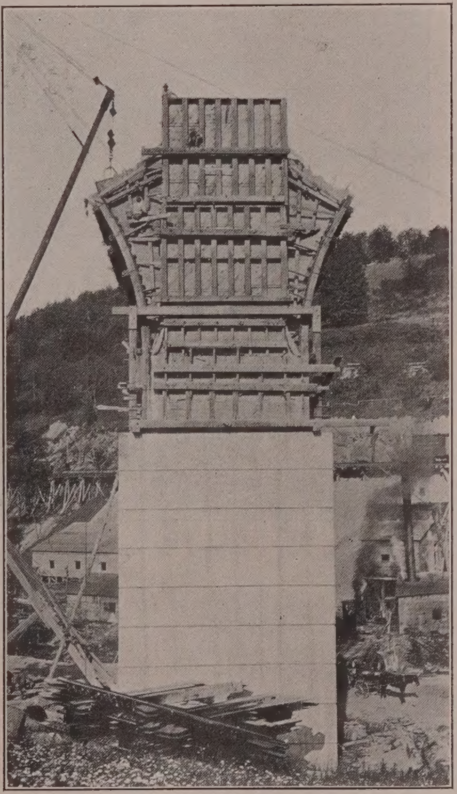
Sheeting, Draining and Concreting

About $10,000 worth of Lackawanna steel sheeting was required for the work, and some of it has been pulled and redriven four or five times, and endures the service so well that about 80 per cent of it will eventually be salvaged. The sheeting was driven with care, and when a pile encountered serious obstruction by a boulder, driving on it was suspended and adjacent piles were driven down beyond the boulder and excavation made to undermine and move the boulder, or in some cases it was blasted and removed so that the piles could be driven without further trouble. By this scheme the sheeting was put down with very little battering.

Although water was encountered, no difficulty was experienced in keeping the foundations dry by the use of 6 and 8-in. centrifugal pumps. Most of the excavation was done by a 1½-yd. Williams bucket suspended from a derrick boom. The sheet piles were covered with tar paper to prevent adhesion of concrete, and the cofferdams were filled nearly to the surface of the ground with concrete deposited against the steel sheeting without the use of forms. After the concrete had set the sheet piles were pulled in sets of one, two or three by an eight-part tackle suspended from an A-frame and by a whip line led direct from the derrick boom to the hoisting engine.

Pier Construction

Above the surface of the ground the pier concrete was placed in wooden forms made of large panels 17 ft. 9 in. high, lined with galvanized iron. After four 4-ft. courses of concrete had been deposited in the form and the last course had set several days, the panels were disconnected, were hoisted 16 ft. by the derrick and reassembled in position for concreting above, and so on. Concrete was distributed to the piers in three-car trains, each taking two full buckets, with room to receive a third empty bucket.



Form for Umbrella Top of Pier

Pier concrete was laid during the coldest winter weather, care being taken to heat the water and the piles of sand and stone and to keep the forms warmed by steampipes under protecting tarpaulins. The maximum amount of concrete deposited in one month was 8500 yd., in June of this year, working one ten-hour shift daily. About 6000 yd. of this concrete were deposited in forms; the average haul was 800 ft.

The concrete for each pier was handled by a guyed-derrick with a boom from 80 to 90 ft. long operated by a Mead-Morrison three-drum hoisting engine, of which sixteen are installed on the work, one for each derrick on the viaduct and three at the concrete plant. The derricks were at first set up on the surface of the ground or on timber towers at the pier site and after the completion of the piers up to the top of the extensions for the arch ribs the derricks placed a ginpole on top of the pier and the latter lifted first the derrick masts and then the derrick boom to new position on top of the pier. The derrick then removed the ginpole and was in readiness to erect the steel truss centers for the concrete arches and to lay concrete upon them.

Arch Construction

The 150-ft.-span arch ribs are built on steel truss centers exactly similar to those described for the Tunkhannock viaduct, but have smaller dimensions. The smaller spans have wooden centers. Four steel centers, each having five ribs or trusses, have been provided and erected on the piers in readiness for the construction of one rib of each of four consecutive spans. After these ribs are completed and the concrete sufficiently set the centers will be struck and the sets of trusses will be moved as units 29½ ft. transverse to the axis of the bridge into the center line of the other ribs which will be built on them, after which the centers will again be struck, the trusses they temporarily supported from the finished ribs separated into four pieces each, lowered to the ground by tackles suspended in the clearance between ribs, transferred to adjacent spans, re-erected, and so on until the spans are all completed.

After the concrete has set thirty days the centers are struck by means of the adjustable members in the crown panels of the trusses, similar to those illustrated in the description of the Tunkhannock viaduct published in the Engineering Record of Nov. 29, and the latter

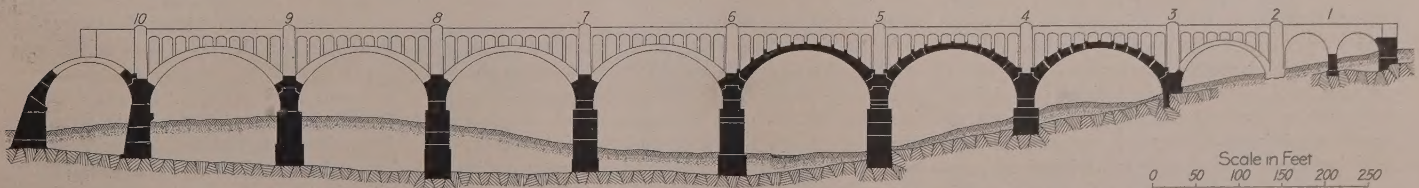


Diagram Showing by Black Areas the Monthly Progress of Construction of Martin's Creek Viaduct are transferred to their new position. Part of the concrete for the arch rings will be handled by the pier derricks. As these cannot reach to complete the spandrel walls and floor arches in the centers of the spans, it is probable that they will be supplemented by a cableway. Both ribs of each 100-ft. span are simultaneously concreted on centering formed of timber trusses, with horizontal bottom chords supported on the umbrella projections from the piers and on a pair of eight-post framed center towers.

Steel Arch Truss Centers

The trusses, which weigh about 28 tons each, are erected four pieces each by the derricks on the piers. The end pieces are pin-connected to the steel grillages on the pier copings 11 ft. below the springing line and are held in position by anchorage bolts passing through the tops of the concrete umbrella sections and by anchor bolts at the foot of trusses. They are self-sustaining until the crown sections are in turn hoisted by the derricks and bolted to them and the center pins driven, making them self-supporting. In each set there are five trusses spaced 3 ft. 9 in. apart and braced together with sway frames and top-and-bottom lateral X-bracing with bolted connections.

The trusses are covered with lagging, of which the upper surface is dressed smooth and greased. Bulkheads are built on the sides and parallel to the axis of the arch to divide the ring into sections corresponding to voussoirs. These are concreted in pairs symmetrically placed on opposite sides of the center, the work being done in the sequence indicated by the numerals on the accompanying diagram, so as to maintain balanced loadings on the centering. After adjacent voussoirs are cast, the bulkheads between them are removed and the narrow portions of arch ring between them are concreted, forming keys which lock them together.

Progress of Work

The erection of the steel centers was commenced July 6, 1913, and the entire 700 tons of it were in position Sept. 25. Concreting of the first arch ring was commenced Sept. 1. About 175 men are employed one ten-hour shift daily, and on Oct. 15 the contract was about 55 per cent completed, and was well ahead of schedule requirements.

The design and construction of the viaduct are under the direction of the engineering department of the Delaware, Lackawanna & Western Railroad Company, of which Mr. G. J. Ray is chief engineer, Mr. F. L. Wheaton engineer of construction for the Martin's Creek cutoff line, Mr. A. B. Cohen concrete engineer, and Mr. Walter Lozier resident engineer in charge of the viaduct. The contract for the viaduct was awarded in June, 1912, to the F. M. Talbot Company, of New York City, of which Mr. Frank M. Talbot is president and Mr. E. J. Maliady secretary and treasurer.

Dogs in Field Service were utilized in the construction of the National Transcontinental Railway. Their use was described recently by Mr. R. F. Uniacke, bridge engineer of the National Transcontinental Railway, at the Canadian Railway Club. Not only were dogs used to transport the mails over the snow-covered trails, but they were also employed in the summertime to operate a light pile driver used with a pump for sinking 4-in. pipes to secure wash-boring tests of the soil in the river bottom. The dogs were hitched to capstan bars of a windlass on a raft on which the pile driver and pumps were installed.

Flood Wall at Portsmouth, Ohio

Design, Cost and Service Record of Reinforced-Concrete Defenses Built in 1908 to Protect City at 62-Foot Flood Stage

By J. I. HUDSON

Formerly City Engineer of Portsmouth

AMONG the first flood walls built for city protection on the inland waterways of this country was the one constructed during the summer and fall of 1908 at Portsmouth, Ohio. The writer, then chief engineer of the Department of Public Service, prepared all the preliminaries in the spring, a contract was let in July, construction was begun on Aug. 1, and the work completed on Jan. 5, 1909.

This work was begun in the face of the most strenuous opposition on the part of many of the best citizens, who condemned the administration, the engineer and the structure in the severest and most intemperate terms. It was not until January, 1913, that these same persons ceased to denunciate, for the protection afforded by the wall to the city during the flood of that month saved several times its cost of construction and kept the city perfectly dry within the flood zone. Even in the flood of March, 1913, it kept out the water until a level of 62 ft. (the top of the wall) was reached, thereby enabling many persons to remove their effects to places of safety.

One of the Temporary Wooden Flood Gates in the Wall

The floods of January and March, 1907, had almost paralyzed the town. The loss of property was large. The people for a time were completely stunned and confidence had to be restored. Accordingly, the engineer was instructed by the council and the Board of Service Directors to construct a flood protection around the west end of the city and along the Ohio River front, a distance of more than a mile. The contract was awarded to Kelly Brothers, of Portsmouth, who vigorously prosecuted the work to completion, losing scarcely a day until the structure was turned over to the city authorities.

With the connecting walls between wing walls and main walls and the wing walls at the driveways there are 6520 lin. ft. of wall. This includes 265 ft. of a temporary wall of plain concrete at one terminus and 722 ft. of plain concrete wall in the west end, terminating in the railway embankment, which constitutes the remainder of the flood defense along the Scioto River as far north as the corporation limits.

The two sections of wall just referred to are smaller and are not of the same type as the wall around the Point at the mouth of the Scioto and along the Ohio River front, being plain concrete walls 8 or 10 in. in width on top with a slight batter on each side; the height varies from 6 to 10 ft., with a footing course possibly a foot wider than the bottom of the wall.

Structural Details

The wall around the Point, 365 ft. long, and along the Ohio River front from Scioto Street to Chillicothe Street 4100 ft. long, is of the inverted T type, with the toe on the river side and the heel on the city side. This wall is composed of four parts—coping, 12 in. thick by 20 in. wide; face plate, 12 in. thick just under the coping and 18 in. thick where it connects with the base plate—this for a section 12 ft. in height from the bottom of the base plate to the top of the coping; base plate, 18 in. thick for all sections, with a width equal to one-half the height of the wall from the bottom of base to the top of coping—this base plate consists of a toe and heel and for a 12 ft. wall the toe is 18 in. in width and the heel 3 ft.; buttresses or counterforts on the city side, spaced 10 ft. on centers, extended from the top of the base plate to the bottom of the coping.

This structure was built to protect the city from a flood stage of 62 ft. on the local gage, which corresponds to about 533 ft. on the government gage. The height of the wall was regulated as closely as possible to conform to the slope of the water at a 62-ft. flood tide, so that as the work progressed eastward with the structure the elevation was raised 0.01 ft. for every 100 ft.

The main wall varies in height from 10 to 22 ft. and the top of the wall is from 5 to 7 ft. above the street level. A structure of this kind must be not only a retaining wall, but, in addition, it must be able to withstand the force of the waves at floodtide when driven in by terrific windstorms. This structure may, therefore, have more mass and more reinforcement than are usually found in ordinary retaining walls, for after the usual calculations are made for the ordinary wall it must also be figured as a structure that will resist the pounding and buffeting of the waves, and parts that are in compression and tension on the one hand become tension and compression parts respectively on the other.

The concrete mixture throughout was 1:2:4. Most of the wall was heavily reinforced. All steel was soft, cold-twisted and figured for a permissible unit compression of 12,000 lb. and a permissible unit tension of 16,000 lb.

Reinforcement

The coping is reinforced with six ⅝-in. longitudinal rods equally spaced, three above and three below, 4 in. from the upper, lower and outer faces. About every 6 ft. a ½-in. transverse rod was hooked over the longitudinal rods in order to keep the latter in place. The face plate is reinforced in the upper half, 4 in. from each face, with ⅝-in. longitudinal rods spaced 14 in. apart on each face or 7 in. apart alternately; and in the lower half are ¾-in longitudinal rods spaced 10 in. apart on each face or 5 in. apart alternately. The vertical rods in this plate are ½ in. spaced 3 ft. apart on each face or 18 in. apart alternately.

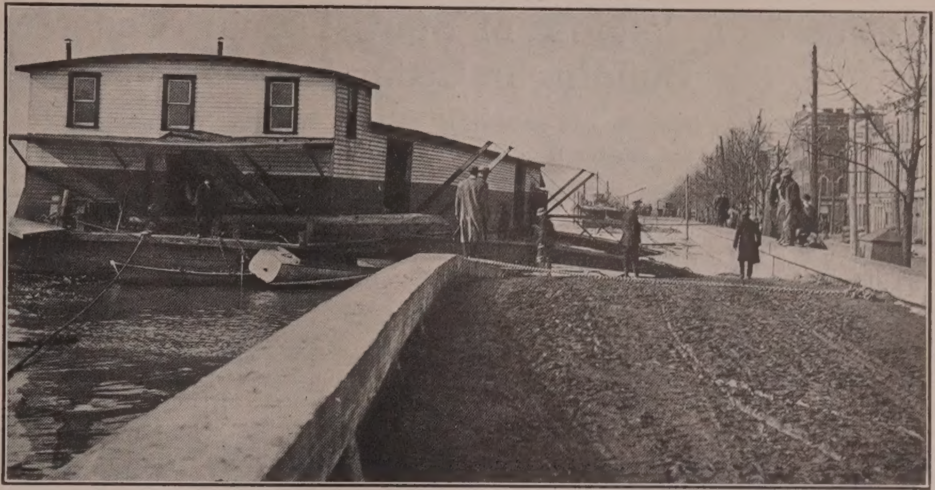
The base plate is reinforced with ¾-in. longitudinal rods arranged in two layers, which rods are spaced 6 in. on centers and 5 in. from each face. The outer rods are 4 in. from the vertical faces of the base plate. The upper layer of ¾-in. horizontal rods rests upon ½-inch transverse rods extending to within 4 in. of the outer faces of the heel and toe. The ½-in. transverse rods are spaced 12 in. on centers.

In the buttresses the reinforcement consists of six 1-in. tension rods hooked around the longitudinal rods in the coping, and in the base plate these are evenly spaced about 4 in. from the outer surface. From every horizontal in the face plate there is a ½-in. tie rod hooked over the tension rods in the buttresses. All the rods throughout were tied together to form, as far as possible, a rigid framework before the concrete was poured.

On the city side of the wall is an embankment varying in width on the top from 5 to 6 ft. The top of this embankment is about 18 in. below the top of the coping. It was intended that a cement walk about 4 ft. wide be laid on this, so as to have the walk and the bottom of the coping at the same elevation.

Costs and Quantities

There were 109.02 cu. yd. of plain concrete at $4; 511 cu. yd. of plain concrete at $4.25 and 4869.55 cu. yd. of reinforced concrete at $6.35—a total for concrete of $33,529.43. Other work—excavation, backfilling, embankment, paving, sewering, etc—cost $4,098.06. Altogether the contractor received $37,627.49. The engineering, drafting, inspecting, typewriting, printing and advertising, stakes, etc., amounted to $1,637.86. The total cost to the city, exclusive of the engineer's services, which were not charged to any particular work, was $39,265.35.



Flood Wall Construction to Allow Trucks to Reach Boat Landing

This low cost may, to some extent, be accounted for by the fact that at the time of construction there was a cement plant in the vicinity. The sand and gravel were within a very short haul, and the cost of every kind of material used on the work was comparatively low. Based upon the prices of to-day, the same work could not be done for much less than $70,000.

This structure has had two very severe tests, and there can be no question of its stability and its usefulness to protect from the flood stages up to 62 ft.

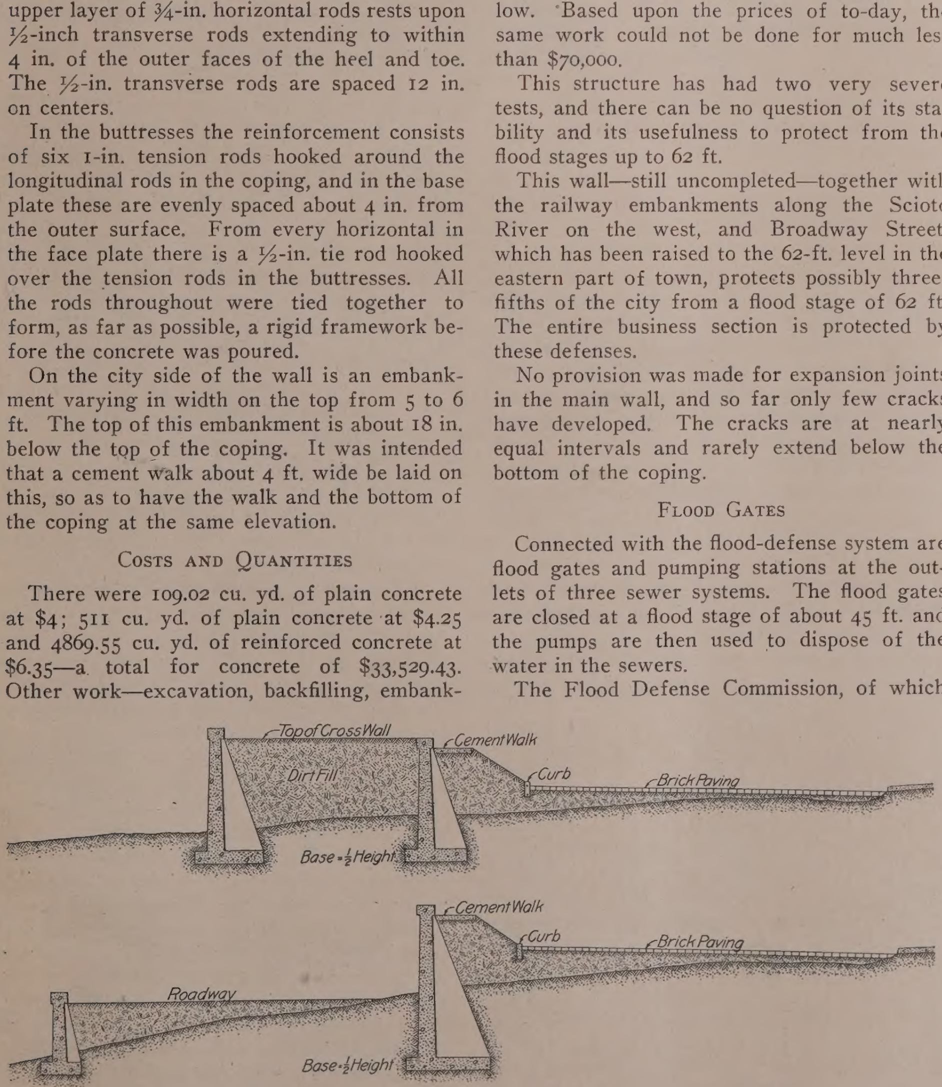
This wall—still uncompleted—together with the railway embankments along the Scioto River on the west, and Broadway Street, which has been raised to the 62-ft. level in the eastern part of town, protects possibly three-fifths of the city from a flood stage of 62 ft. The entire business section is protected by these defenses.

No provision was made for expansion joints in the main wall, and so far only few cracks have developed. The cracks are at nearly equal intervals and rarely extend below the bottom of the coping.

Flood Gates

Connected with the flood-defense system are flood gates and pumping stations at the outlets of three sewer systems. The flood gates are closed at a flood stage of about 45 ft. and the pumps are then used to dispose of the water in the sewers.

The Flood Defense Commission, of which the writer was a member, appointed in 1900 by the city council, planned this line of defense and made a report of surveys, plans and estimates to the city council. The report was based upon earth levees estimated to cost about $125,000. For a part of this sum bonds were sold and the work begun. At that time no concrete walls were considered in this line of defense. From time to time for a number of years the city of Portsmouth has been proceeding along the lines mapped out by the commission about thirteen years ago. Should the concrete wall be substituted for earth levees all along the line, it would, of course, cost more, but it would be safer, the sandbag brigades would be entirely eliminated, and all anxiety caused by the washing and breaking of earth levees would be dispelled.



General Arrangement of Flood Walls at Portsmouth

Main Drainage of London

It is now nearly 60 years ago since it was determined to provide for the complete interception of the sewage of London, and for its discharge into the River Thames below the Metropolis. The old sewers still remain for storm overflows direct into the river. The works then adopted required 18 years for completion, and consisted of the construction of intercepting sewers parallel to the course of the river, and connected to the old main sewers. The dry weather flow provided for, which was for a population of 3,500,000, was 108,000,000 gal. per day, and 286,000,000 gal. of rainfall, but the discharging capacity of the sewers was made much larger than this quantity in view of the fluctuations in the rate of flow. In 1901, to meet the increased demand, additions were put in hand, and up to the present, when completion is shortly to be effected, there have been added to the mileage of the sewers 44 to the north system and 33 to the south, mostly main intercepting sewers, while about 10 miles of storm water relief sewers were put in. Altogether, since 1850, 370 miles of mains have been constructed.

Discrepancies in Old Surveys are being found by the Forest Service of the U. S. Department of Agriculture in surveying mountain lands in the Appalachians for Government purchase. Taking the regions as a whole, the acreage surveyed by the Government exceeds by 1 per cent the acreage claimed by the owners, but in certain instances the excess is much greater. One tract in Tennessee, supposed by the owners to contain 850 acres, actually contains 1480 acres.

Causes of Collapse of Stark-Lyman Concrete Building in Cedar Rapids

Excerpts from Report Submitted by C. A. P. Turner, of Minneapolis, to the Master Builders' Association of the State of Iowa

FOLLOWING the collapse of the Stark-Lyman seven-story reinforced-concrete building in Cedar Rapids, Iowa, on Nov. 14 the Master Builders' Association of the State of Iowa selected Mr. C. A. P. Turner, of Minneapolis, to study and report the facts and conditions relative to the accident. (See Engineering Record, Nov. 22 and 29.) His report was submitted on Dec. 6 and from it the following excerpts are taken:

Type of Building

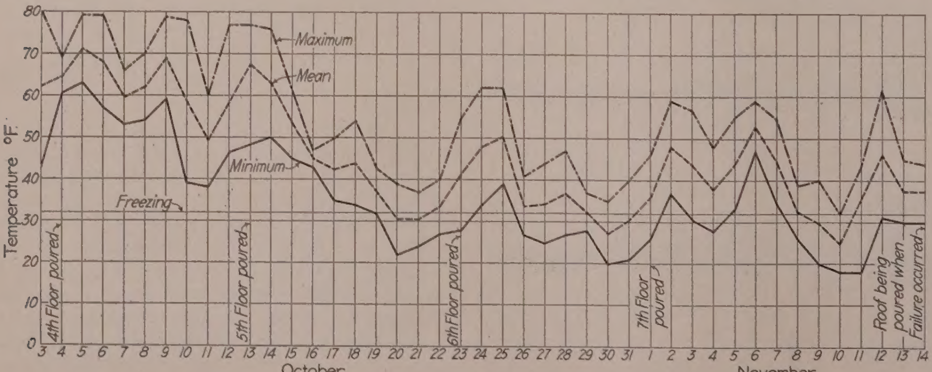
The building was not strictly a reinforced-concrete building. It was so only in part. It presents a combination of hollow tile, concrete and steel. The floors throughout were made up in a large part of tile with thin narrow joists of concrete between the separate tiles and a layer of 2 in. of concrete covering the tile on top, these joists being each reinforced with two twisted rods. The Stark-Lyman building presents only the latest instance of an almost interminable series of similar catastrophes with the identical combination of tile, concrete and steel. This type of construction, involving the inherent risk of erection, was not selected by the contractor, but was selected for and on behalf of the owner by the architect, and forced upon the contractor by the purchase of the system by the owner, and the appointment by the owner of an engineer to superintend the construction, and, presumably, if the engineer were competent to shoulder the risk or hazard of erection, by giving competent, proper and needful instructions for safely carrying on and executing the work.

The tile used in the upper floors was 7 in. thick by 12 x 12 in. approximately. These tile were moderately hard burned, with a thin ½ to ⅝-in. shell, and, like all tile of this character, if one of them should be dropped on the pavement it would break into small pieces. This characteristic of fragility and brittleness of each individual block under any shock or blow is equally a property of the entire floor.

Analysis of Design

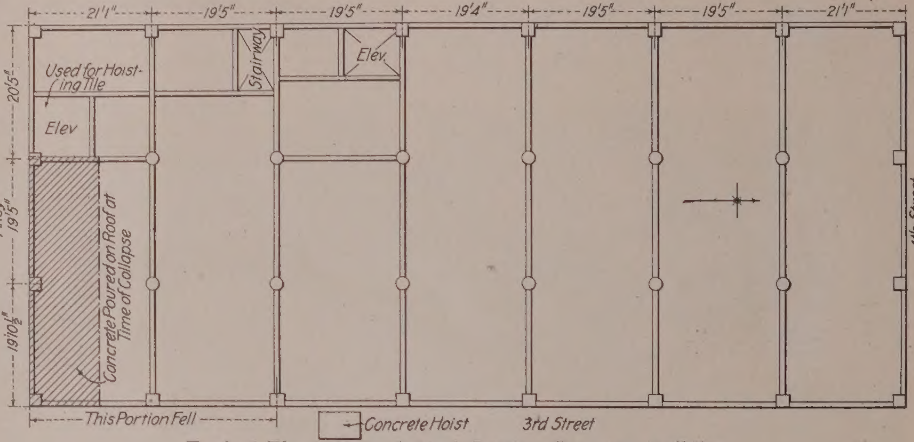
Reinforcement.—The reinforcement consisted of square twisted bars, which showed by the manner in which they were bent that the material was tough and of reasonably fair quality.

Concrete.—A careful examination of the broken concrete indicated beyond question that a liberal amount of cement had been used throughout and that the concrete itself was of unusually good quality. The sand found on the job was exceptionally good. The rock was of fair grade. There was noted this striking characteristic in nearly all fractures of concrete observed—all presented a clean-cut cleavage of the stone aggregate, including those on the fourth and fifth floors.



Ranges of Temperature during Period of Concreting

General Details.—The details of the floors may be characterized as a combined beam-and-slab construction in which the beams run crosswise of the building and the slab spans from beam to beam. The reinforcement is a one-way type and the joint between the tile is nominally 4 in.; but, as is usual with this kind of construction, the alignment of the tile varies somewhat and the width of the joint varies with the alignment. There are a number of places where there are slight imperfections in the casting and pockets are formed, though not to such an extent as seriously to impair the degree of strength ordinarily secured by the arrangement adopted. This general type of one-way beam and slab construction, whether of reinforced concrete or a combination of tile and concrete, is the least conservative arrangement that it is possible to devise, since failure with this type is as sudden and as complete as can be secured by the most ingenious arrangement of materials that it is possible to devise, were this the specific object in view.



Typical Floor Plan of Stark-Lyman Concrete Building

Column Construction.—The lower tiers of columns are of the hooped and vertically reinforced type. This hooping is stopped on most columns as low as the fourth tier and on some lower than that, and from this point upward the columns are reinforced in the main with either four or eight 5/16 to ⅜-in. vertical rods, with tiers 12 or 15 in. apart.

A hooped column, with hoops binding the core and holding it in shape under excessive load, or when the concrete is not thoroughly cured, renders this type tough, reliable and safe. The second type of column, with vertical reinforcement and tiers at intervals of 12 to 14 in., has little or no toughness—the concrete body is not well bound together, so that unless the concrete is well cured and hardened sudden collapse and failure may readily occur without any warning whatever.

Beam Details.—Some of the most important beams in the structure were provided with no stirrups whatever, the reinforcement at the bottom and top of the beam being parallel to the center line for some distance from the support. The reinforcement which was carried up to the top of the beam was carried out a considerable distance from the support, and then bent down sharply, affording in this unscientific position little or no assistance to the concrete in shear at the time when it needs this assistance most—that is, in the partly cured condition. This radical departure from good practice, in leaving out stirrups and improper bending of beam rods, we find in the important member, beam 41, into which beam 33 frames as offset in the upper stories by column 18.

Temperature Conditions

The dates of pouring concrete were as follows: Fourth floor, Oct. 4; fifth floor, Oct. 17; sixth floor, Oct. 23; seventh floor, Nov. 1. The roof was being poured on Nov. 14, and part of the first bay on the rear end was in place when the accident occurred.

By reference now to the temperature chart it will be noted that about Oct. 13 the fifth floor was poured, and that the minimum temperature at this time was commencing to run below 50 deg. Fahr. The dates given are probably those on which they commenced to pour this floor, and following the line of minimum temperature we see that it drops rapidly from 50 to 21 and 22 deg. above zero on Oct. 20, before this floor was presumably finished and work started on the next floor. The concrete, contrary to good engineering practice, was mixed with cold water, and hence the hardening or curing of the concrete so cast must have been very slow indeed. This would be particularly true of the concrete in the column boxes. That which was cast in the slab, on account of the absorption of the excess moisture by the tile, would harden with much greater rapidity, and might be expected to develop a reasonable compressive resistance between this period and the period at which the collapse occurred, especially in view of the fact that from Oct. 22 to 26 the maximum temperature rose to 60 deg. While the concrete mixture was of the right consistency and strength, it was not of the right temperature to properly harden in the range of temperature shown by the chart.

Specific Causes of Failure

In beam 31, framing into column 18, on the upper floors, we have a weak member, deficient in reinforcement where beam 33 frames into it. We find the shear on the concrete in this beam in this partly cured condition higher than is ordinarily considered permissible with old and cured concrete. This fact, combined with the unquestionably soft condition of the concrete, due to the chilly weather, and the fact that the column reinforcement where beam 41 frames into it at the fifth floor and at the top of the fourth tier of columns above the basement consists merely of vertical rods with ties at considerable intervals apart, makes this point, in the writer's judgment, the initial point of failure. The giving away of the column at this junction of the beam and column would cause a wrenching through beam 35 of column 15 and the almost instantaneous collapse of the two end bays of the building. This conclusion is in accord with the manner in which the failure occurred and seems substantiated by what evidence has been given by various witnesses.

It has been suggested that settlement of the supports may have had something to do with the failure. Careful levels were taken upon the center line of columns in the section which collapsed and no change of level was discovered. It is impossible to account for the collapse on this ground. The fact that the failure was instantaneous, or nearly so, would preclude the possibility that failure of the centering was the cause of the collapse, as has been suggested by those who maintained that the small amount of concrete which had been poured on the roof above the seventh floor over an insignificant area may have produced the collapse through failure of the supporting forms. A collapse of this character would be accompanied by a kind of step-by-step failure, as was noted distinctly in the collapse of the Long Beach Hotel, where the upper floor came down first and one floor after another followed. In the case under discussion, however, this phenomenon was absent and a careful consideration of all facts seems to determine with a high degree of certainty the manner of failure as here advanced.

The contractors put up a better building and a stronger building than the plans called for. They made the following change in the plans: They proposed using round columns, expecting that there would be some economy in the interior columns, but after having secured permission to make this change they wanted to go back to the square form shown on the owner's design, but the engineer for the owner is credited with having objected to this change back to the original plans. Accordingly, we find at the weak point above noted—the top of the column under the fifth floor, at beam 41—an increase in cross-section of the concrete of fully 20 per cent over and above the cross-section required by the engineer's design as adopted by the owner, and figured on by the contractor, and a corresponding increase in the strength of the structure at this point at the period of failure of not less than 35 or 40 per cent, taking into consideration the beam and column connection. In spite of this the collapse has occurred, and undoubtedly through failure at this point in the design after the contractor had increased the strength of the structure at the period of collapse at least 30 to 40 per cent over and above the degree of strength provided by the engineer's design, and notwithstanding all this, the coroner's jury placed the blame of this collapse squarely on the shoulders of the contractor.

Cause of Cave-In of Randolph Street, Chicago

Abstract of a Report Presented by a Commission of Municipal Engineers, Consisting of John Ericson, George Weston and John A. Dailey

CONCLUSIONS as to the cause of the cave-in of Randolph Street, Chicago, on Dec. 2 (see Engineering Record, Dec. 13, page 677) have been reached by a commission of three engineers appointed by Mr. L. E. McGann, commissioner of public works, to report on the accident. These conclusions follow:

First, the primary cause of the failure of certain parts of the building structure and consequent settlement of the street was the insufficient temporary bracing of the steel sheet piling during the excavation for and the construction of the basements of the building.

Second, in this construction, where the steel sheet piling is designed to carry floor loads, adequate footings of sufficient area to support the superimposed loads should be provided. The plans do not show any such footings or supports for the sheet piling, but such footings are provided at the Washington Street section of this building, and it is stated that they were omitted on these plans by mistake.

Third, the floor design of the basement and sub-basement floors on the Randolph Street side, for the beam framing around the openings of the fresh-air intake and stair wells, and the framing between these openings and the steel sheet piling are inadequate to safely brace the sheeting against the earth pressure.

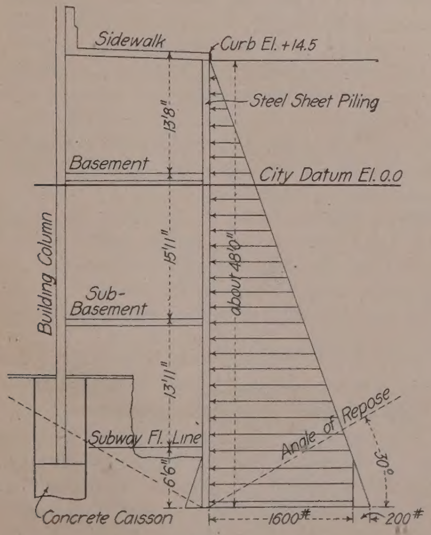
Fourth, neither the city water main nor the sewer was the cause of this failure.

Investigating Commission

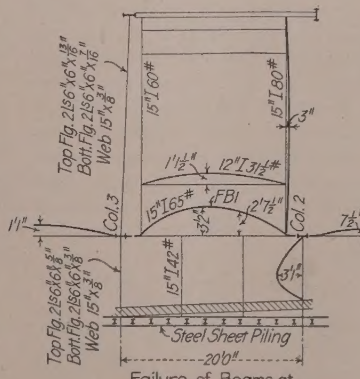
The commission was appointed to consider the cause of the damage to municipal and public-utility properties in streets by reason of street settlements adjacent to buildings under construction. In the Randolph Street case members of the commission made a personal inspection the morning following the accident and appointed two engineers, Messrs. Hugh E. Young and A. B. Callander, representing Mr. John Ericson, city engineer, and two representing Mr. E. B. Weston, of the Board of Supervising Engineers, Messrs. H. C. Lummis and H. T. Cooper, to obtain measurements and data required for an analysis of facts and conditions contributory to the settlement of the street and resulting damage.

Following a detailed description of the failure, which is substantially the same as reported in the Engineering Record of Dec. 13, except that the commission found that the trench excavated along the sheet piling had been carried much lower than reported) is an analysis of the contributory causes. It is assumed, says the report, that this trench had been dug in order that the portion of the subway floor between the columns and the sheeting could be built.

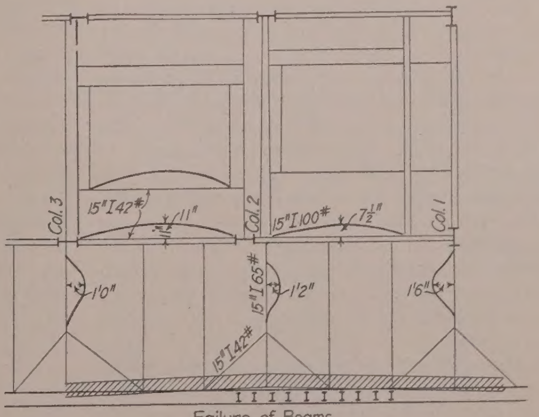
The steel sheet piling had been driven about 48 ft. below the curb. The excavation for the building proper had been carried to about 5 ft. below the sub-basement floor, and a trench had been dug adjacent to and to within about 6 ft. of the bottom of sheeting. The sidewalk floor, the basement floor and the sub-basement floor were in place, but the subway floor had not been built. The entire lower portion of the sheeting, a distance of about 19½ ft., was unsupported except for about 5 ft. of unexcavated material. This part of the sheeting is



Earth Pressure Diagram



Failure of Beams at Sub-Basement Floor Level



Failure of Beams at Basement Floor Level

Earth Pressure Diagram and Sketches Showing How Beams Buckled after Randolph Street Cave-In

subjected to the maximum lateral earth pressure, as shown in one of the drawings.

Inadequate Earth-Pressure Provisions

The following assumptions made in this investigation are considered by the engineers very conservative. Angle of repose of the earthfill in the street was taken at 30 deg. with the horizontal. Weight of the earthfill was assumed at 111 lb. per cubic foot. The effect of hydrostatic pressure and the superimposed loads—such as live loads and the weight of adjacent buildings on floating foundations—was not considered. Hydrostatic pressure probably existed below datum. It is the purpose, states the report, to show that even under the most favorable and conservative assumptions the conditions of construction at the time of failure were entirely inadequate to withstand the earth pressure. The resultant earth pressure per foot of wall was found to be 42,500 lb. acting at a distance of 3.3 ft. below the level of the sub-basement floor. This causes a load of about 54,000 lb. per linear foot on the sub-basement floor. A sketch herewith shows a panel of the sub-basement floor. The beams at this level were subjected to the maximum earth pressure. The heavy lines indicate approximately how the beams failed.

Calculations show that there would have been sufficient lateral pressure on beam FB1 to have caused a fiber stress of about 3,000,000 lb. per square inch. As soon as this beam, which is between columns 2 and 3, buckled, the entire horizontal thrust at this floor was imposed on the 15-in. I-beams between the building columns and the steel sheeting. The plans show that these beams are not normal either to the sheeting or to the columns in the plane of the floor (the horizontal plane). The angle at which they frame is about 52 minutes from the normal. This eccentricity would produce a bending moment in the beams which would reduce the compression resistance. Calculations indicate that there would have been sufficient pressure on the sub-basement floorbeams between the columns and the sheeting to have caused a stress 7.5 times their ultimate strength.

Beam Failures

A sketch of two panels of the basement floor indicates how these beams failed. After the beams of the sub-basement level failed, the load carried by the sheeting was thrust upon the first basement floorbeams with more or less impact. This would cause the upper end of the sheeting to push out toward the street, which movement probably caused the buckling of the pavement and the shearing of the rivets connecting the sheeting to the sidewalk floorbeams.

Calculations show that after the failure of the sub-basement floorbeams the load on the basement floorbeams, due to the earth pressure, would have caused a stress in these beams of 3.1 times their ultimate strength, but the sheeting itself could not have transferred this load; therefore, calculations were made to show that the load, which could be carried by the sheeting at its ultimate strength, would, however, produce a stress in the beams 1.34 times their ultimate strength. This does not include the additional stress due to the eccentricity of the load on the beams, which increases the stress to about 3.9 of the ultimate strength.

The vertical load of the steel sheet piling, including the concrete which incases it from the sidewalk level to the sub-basement floor and the weight of the floor hanging on the sheeting, amounts to about 12,000 lb. per linear foot. Assuming a frictional resistance of steel on mud of $\frac{1}{2}(0 + 1800)\ 0.333$ (Coefficient of friction) $= 300$ lb. per square foot, the total frictional resistance per linear foot of sheet piling would be about 14,400 lb.—a factor of safety of 1.2. In other words, the frictional resistance of the piling against the fill would be sufficient to support the vertical loads on the sheeting at the time of the failure. In determining the bearing value of the sheet piling under the dead and live loads of the sidewalk, basement and sub-basement floors, the calculations show that the sheet piling would sustain only about 0.685 of these loads. When sheet piling is used to carry floor loads, as in the case of this building, the report states, it should have adequate footings, consisting either of caissons carried to hardpan or rock, or spread footings of sufficient area to support the superimposed loads. Another form of construction for supporting the floor loads would be to carry the loads to the main building columns by bracing. The frictional support of sheet piling by mud or wet clay—which is the material usually found below datum—is too uncertain a quantity to be depended upon for supporting permanent dead loads and varying live loads.

Conclusions

The following conclusions were arrived at by the four engineers: First, the lateral movement of the earth was primarily caused by the excavation of a trench adjacent to and along the entire length of the sheeting at the subway level. This excavation was made without sufficient temporary bracing being placed to take the resulting earth pressure, both at the sub-basement and subway levels of the structure.

Second, as a consequence of this insufficient bracing of the two lower floors, the I-beam struts at the sub-basement level, which supported the sheet piling, failed, allowing the sheet piling to move in. After these beams had failed, the load was transferred to the bracing in the basement floor, which in turn failed, rupturing the connection of the sidewalk floor to the sheet piling.

Third, the movement of the sheet piling in toward the building caused a displacement of the earth to occur, which resulted in a settlement of the street. A computation of the comparative yardage contained in the depression of the street below normal level and the displacement of the piling resulted in 300 cu. yd. for the former and 265 cu. yd. for the latter. This downward movement of the earth carried the sheet piling with it. The sheet piling, which was already loaded to almost its full frictional resisting power, in this construction constitutes a retaining wall and also a support for the floor loads. The investigation shows this retaining wall should be thoroughly supported by temporary bracing while the permanent floors are being constructed, liberal allowance being made at all times for the horizontal thrust of the earth.

Engineering Problem of Electrification

Abstract of a Paper Presented before the Canadian Society of Civil Engineers on Dec. 18 by A. H. Armstrong, of the General Electric Company—Summary of Discussion

At the regular monthly meeting of the Canadian Society of Civil Engineers in Montreal, Dec. 18, Mr. A. H. Armstrong, of the railway department of the General Electric Company, offered data to prove the superiority of high voltage direct current system to alternating-current systems of split-phase or other types. In the discussion which followed Mr. George Gibbs announced that the Pennsylvania Railroad had decided to adopt alternating current for its suburban electrification at Philadelphia, while Mr. Armstrong stated that the Chicago, Milwaukee & Puget Sound would use 2400-volt direct current for its 440-mile trunk-line electrification in Montana. An abstract of Mr. Armstrong's paper follows:

It is proposed to replace the steam engine with a type of motive power that offers superior advantages in the hauling of heavy trains. In other words, the electric locomotive itself constitutes the main argument in favor of electrification and no marked excellence of distribution system can offset the failure of the electric motive power. The steam locomotive it is proposed to replace is a highly developed machine of great reliability and the result of the experience born of a great many failures. It cannot be too strongly emphasized, therefore, that the electric motive power is the controlling factor in main-line electrification, a point of view that is sometimes overlooked.

Three Systems Considered

The three electric systems considered for main-line electrification are as follows: Single-phase, alternating; split-phase, alternating, and high-voltage, direct current.

It has been found that the initial expense and cost of upkeep of rolling stock equipped with single-phase commutating motors is fully double that of cars having the same seating capacity and equipped with direct-current motors.

The introduction of the single-phase system was a result of the success of suburban and interurban electric railway operation and the extension of these lines over large areas, thus bringing into prominence the question of economical power distribution. It was recognized that a voltage higher than the commonly accepted standard of 600 volts was desirable upon the trolley in order to minimize the cost of installing feeder copper and substations. While the single-phase motor was being developed and installed upon interurban railways, careful attention was also being given to the question of the possibility of using d.c. motor equipments at higher voltages, and this resulted in the installation of the first 1200-volt road, the Indianapolis & Louisville Traction Railway, operated in 1907. The success attending this operation led to other similar installations at both 1200 volts and 1500 volts until it is now generally recognized that the high-voltage d.c. system is without a competitor for all classes of suburban and interurban electric railways.

Steam Railroad Electrification

The history of the battle of the systems and the elimination of the single-phase motor as being unsuitable for the equipment of light electric railways has an important bearing upon the selection of systems for main line electrification. The limitations of the single-phase motor that lead to its failure in the interurban railway field do not appear to be lessened when considering it for locomotive

Table 1—Main-Line Electrification—United States and Canada

Installation	Year	Type Locomotive	System	Voltage
St. Clair Tunnel	1908	Geared	Single-phase alternating	3,300
N. Y., N. H. & H.	1907	Gearless	Single-phase alternating	11,000
Hoosac Tunnel	1911	Geared	Single-phase alternating	11,000
Cascade Tunnel	1909	Geared	Three-phase alternating	6,600
*Norfolk & Western	1914	Geared Side Rod	Split-phase alternating	16,500
Baltimore & Ohio Tunnel	1895	Geared	Direct current	600
New York Central	1906	Gearless	Direct current	600
Detroit Tunnel	1910	Geared	Direct current	600
Pennsylvania Terminal	1910	Side Rod	Direct current	600
Butte, Anaconda & Pacific	1913	Geared	Direct current	2,400
*Canadian Northern	1914	Geared	Direct current	2,400
*Canadian Pacific	1914	Geared	Direct current	2.400

*Under construction.

equipment, with the result that it is in use but on three of the twelve roads that are truly representative of electrified steam roads operating large electric locomotives. (See Table 1.)

Split-Phase System

It is a noteworthy fact that the use of the single-phase motor has not extended beyond the two original roads installing this type of equipment, the Grand Trunk and the New York, New Haven & Hartford (including Hoosac Tunnel installation), whereas direct-current motors have been universally adopted in all the more recent electrifications with the single exception of the proposed split-phase installation on the Norfolk & Western Railway.

[In this system the line current is single phase. By means of a "phase-converter" or "phase-splitter" polyphase current is produced and supplied to the main motors, which are of the induction type.]

The so-called "split-phase" system is comparatively a newcomer in the electric traction field and it has not yet been subjected to the test of actual operation. The proposed system offers many attractive features, however, and it is worthy of careful study in order to understand its fitness for heavy electric railway service. From experimental tests made, it seems reasonably certain that the split-phase locomotive can meet the demands of commercial operation with satisfactory reliability.

Direct-Current System

Confronted with the problem of main-line electrification and the demand for a distributing system which would provide for the economical distribution of large units of power over an extended area, the need of higher direct-current voltage was appreciated and resulted in the first installation of 2400 volts direct current upon the Butte, Anaconda & Pacific Railway, first operated May 28, 1913. This installation marks an epoch in electric railway progress, as its success offers substantial proof that direct-current motor equipments can be constructed at a reasonable cost and operated in an efficient and reliable manner with trolley potentials as high as 2400 volts.

The success of high-voltage d.c. installations has not been marred by a single instance of failure due to fundamental defects in the type of apparatus used, and justifies the conclusion that the 2400-volt, 1200-volt and the 600-volt d.c. equipments are all part of the same general d.c. system, and that the only difference is the need of more insulation in one case than in the other.

Fully appreciating the grave responsibility of selecting a type of motive power for a proposed electrification that holds promise of special fitness for the immediate service contemplated and also is capable of meeting the demands of unlimited future extensions, this paper will briefly touch upon the comparative characteristics of the split-phase and 2400-volt d.c. systems of operation. The choice seems to lie between split-phase and direct current, inasmuch as the history of the single-phase motor equipment does not seem to justify its further consideration for heavy electric railroading.

Starting first with a comparison of the two types of locomotives, it is necessary to make some general assumption as regards service conditions in order to draw conclusions as to relative locomotive characteristics. The electric locomotive is capable of being constructed in very large units, but convenience in shopping and simplicity in construction both point to a unit of approximately 100 tons total weight on drivers. These 100-ton units can be coupled together and operated as a single locomotive of any desired capacity, though probably a two-unit locomotive weighing 200 tons and giving a starting tractive effort of 120,000 lb. is as large as the draft gear will stand.

Experience with steam locomotive practice seems to point to a locomotive rating on ruling grade that calls for a tractive effort corresponding to approximately 18 per cent coefficient of adhesion on drivers. Thus the conditions obtaining upon a 2 per cent ruling grade will be as follows:

Tractive effort due to 2 per cent grade	40 lb
Tractive effort due to train resistance	6 lb.
Tractive effort due to curve resistance	2 lb.
Total per ton	48 lb.
Total weight on drivers	200.000 lb.
Rating at 18 per cent coefficient of adhesion	36,000 lb.
Gross train weight	750 tons
Trailing train weight	650 tons
Speed	15 m.p.h.
Net output at drivers	1,450 hp.

A maximum load of 50,000 lb. per axle has been assumed as being within acceptance of good practice. The question of speed on ruling grade is one subject to local requirements, but in general a speed of 15 miles per hour on 2 per cent grade is as high as desirable on roads of moderate tonnage.

As ruling grade generally extends unbroken over comparatively short distances, it is possible to take advantage of this fact in electric locomotive design and proportion the motive power for a continuous capacity of say 16 per cent coefficient of adhesion without danger of exceeding safe temperature limits in operation. The continuous capacity of the 100-ton unit would therefore be 32,000 lb. tractive effort at slightly more than 15 miles per hour, based upon 16 per cent coefficient of adhesion of the weight upon the drivers.

Weights of Locomotives

Owing to the moderate speeds at which a freight locomotive will operate, it is entirely feasible to consider a construction wherein the motors are geared direct to the driving axles by twin gearing, in this respect following the practice of the Detroit Tunnel, Cascade and Butte locomotive which has proved very successful.

Table 2—Comparative Weight of Locomotive—Continuous Capacity

32,000 lb.—15 m.p.h.

	Split-Phase	2,400 Volts Direct Current
Four motors	44,000 lb.	44,000 lb.
Control apparatus complete	17,000 lb.	27,000 lb.
Air compressor	4,000 lb.	4,000 lb.
Air brake equipment	3,000 lb.	3.000 lb.
Miscellaneous	2,000 lb.	2.000 lb.
Phase converter	22,000 lb.	....
Transformer	18,000 lb.	....
Cab and running gear	160,000 lb.	120,000 lb.
Total	270,000 lb.	200,000 lb.

For the purpose of this comparison, it is assumed that both split-phase and direct-current locomotives will be of similar construction and employ twin geared motors of equal weight and efficiency. A comparison of weight distribution in the two types of locomotives is presented in Table 2.

Efficiency of Locomotives

It is evident that the split-phase locomotive is not only considerably heavier for equal capacity, but also more complicated and inefficient than the direct-current locomotive. The power from the trolley must in turn pass through transformer, phase-converter, control, motors and gears. The efficiency of the split-phase freight locomotive works out to 75.7 per cent on the ruling grade and 70.5 per cent on the level; the corresponding figures for the direct current freight locomotive are 86.6 and 82.0 per cent. Average efficiencies for the two types on grade and level are 73.1 and 84.3 per cent respectively.

Table 3—Efficiency of Distribution System—Detailed Comparison

	Split-Phase	2,400 Volts D.C.
Step down transformers, per cent	97.5	96.5
Motor generator sets, per cent	87.0	81.0
Step up transformers, per cent	97.5	....
Railway transmission line, per cent	96.0	96.0
Line transformers, per cent	96.0	....
Protective transformers, per cent	96.0	....
Trolley, track and feeders, per cent	96.0	88.0
Combined efficiency, per cent	70.5	66.0

For passenger service the efficiency figures given by Mr. Armstrong are as follows: Split-phase, 69.3 per cent on ruling grade, 54.5 on the level, and 61.9 per cent as the average on grade and level; direct current, 85.0 on ruling grade, 89.2 on the level and 87.1 as the average on grade and level. These figures differ from those for the freight locomotive because for the high speeds a different type is assumed to be used. The direct current locomotive would be gearless, while the split-phase would be of the geared side-rod type.

The efficiency of the distribution systems is summarized in Table 3, and the total efficiency of locomotive and distribution system in Table 4.

Table 4—Total Efficiency—Distribution System and Locomotive

	Split-Phase	2,400 Volts D.C.
Freight Service—		
Distribution, per cent	70.5	66.0
Locomotives, per cent	73.1	84.3
Combined efficiency, per cent	51.5	55.7
Passenger Service—		
Distribution, per cent	70.5	66.0
Locomotives, per cent	61.9	87.1
Combined efficiency, per cent	43.6	57.5

This paper is largely devoted to a comparison of alternating-current and direct-current motor locomotives, as lack of appreciation of the fundamental facts involved has perhaps been the basis of the false hopes raised as to the possible advantages resulting from the installation of the single-phase trolley. It surely does look attractive to install a system employing 15,000 volts on the trolley, no feeder copper and no rotating substation apparatus. But investigation and experience disclose the fact that the single-phase trolley is a decided menace to neighboring circuits.

Discussion

In the discussion which followed the presentation of the paper, Mr. H. H. Vaughan, assistant to the vice-president of the Canadian Pacific Railroad, was the first speaker and referred to the adoption of high-tension direct current for a section of the Canadian Pacific. This installation involved 30 miles of route and 43 miles of track with a long 4 per cent grade, the operation being carried on by four locomotives. The battle of the systems, he said, had deterred steam railroad men from adopting electricity for motive power; but in the case cited both systems were found to be practically alike in first cost and in maintenance, direct current being chosen because it was considered more advantageous to use a system in which the larger part of the investment was involved in the cost of copper for feeders, rather than in locomotives which might soon become obsolete. In addition the development of the gearless locomotive and the improved methods of collecting large currents had helped toward the decision in favor of direct current.

Mr. Vaughan thought that the train-mileage cost basis was more reliable than the ton-mileage basis except in case of mountain grades where the amount of coal consumed per train was fairly steady. He said that in the case stated the cost of steam operation was 40 cents per train mile and that of electric operation was only 25 cents per train mile. Repairs amounted to 5 cents per locomotive mile instead of 7½ cents, and with electric power the cost of coal was cut in half, three electric locomotives doing the work formerly done by four steam locomotives. Reduced to a unit basis there was a saving in operating cost of 15 cents per train-mile and this was balanced against the added investment for electrical equipment. He stated that the operation of at least twenty-two trains per day was required in order to make an electrical installation pay for itself. On the Canadian Pacific electrified line, he said, it had been found desirable to minimize the weight on electric locomotive axles, the pressures being kept down to 50,000 lb.

Views of Mr. Gibbs

Mr. George Gibbs, of Gibbs & Hill, consulting engineers, New York, contributed a written discussion. It is in part as follows:

"Reference is made to electric traction in such a way as to lead one to infer that heavy mainline traction is the subject of the discourse. Following this the question of system is taken up, and the single-phase system is promptly dismissed as unsuitable, apparently from unsatisfactory results on a number of interurban traction lines which found it advantageous to change over from single-phase to direct current. Probably these roads never should have used single-phase because their conditions did not suit this form of power, and, in any event, their case is not parallel to that of heavy steam traction.

"On another page the following conclusion is advanced: 'It is a safe prediction to make that no more single-phase motor equipments will be placed in operation in this country on new roads unless these roads virtually form an extension of existing systems.' I take it from the context, although the sequence of the argument is not entirely clear, that this means that no more city or suburban roads will be thus operated; otherwise the Pennsylvania Railroad Company's adoption of the single-phase system motor-car operation on its suburban lines out of Philadelphia places the 'safe prediction' above referred to in some jeopardy.

"After thus summarily dismissing the single-phase motor, the writer then proceeds to advocate high-tension direct current, apparently for all classes of work, and appears to think the split-phase locomotive its only serious competitor. Of course, the fact that single-phase contact and split-phase locomotives were adopted on the Norfolk & Western Railway for the heaviest electric traction work yet undertaken anywhere requires some explaining away, because there are presumably reasons to believe that the adoption of this system was made after some study of the local situation and the conclusion arrived at that the system had preponderating advantages for the work in question.

New York Central's Experience

A letter was then read from Edwin B. Katté, chief engineer of electric traction New York Central & Hudson River Railroad. An abstract follows:

"In glancing over the paper, on the first page I notice that Mr. Armstrong confines his remarks to an electric locomotive capable of hauling an 800-ton passenger train at 60 miles per hour. Such a locomotive is too small to be seriously considered by steam railroad men operating important trunk lines. The modern electric passenger locomotive must have sufficient capacity to haul a train of cars weighing from 1000 to 1200 tons. Please note I say 'train of cars.' Electric companies frequently confuse railroad men by including the locomotives when they refer to a train of a given weight. Further, such a locomotive should be capable of performing the regular service practically continuously and not after a few hours of operation be laid up in the terminal from one to three hours to allow the motors to cool off. Mr. Armstrong also tells you that the electric locomotive requires no stops for fuel or water. This is true in the summer time, but passenger trains have to be heated in the winter, and the locomotive must be taken out of service every few hundred miles for fuel oil or at least water.

Experience of the New Haven

Mr. W. S. Murray, consulting engineer, New Haven, then presented a series of lantern slides showing work on the New York, New Haven & Hartford Railroad and also a series of curves for a proposed 120-mile electrification to show at what point single-phase became advantageous and also the effect of voltage drop with given wire sections under increasing tonnages. He then referred to Mr. Armstrong's paper, saying that he believed it was the strongest card yet played in favor of single-phase traction. There had been too much play upon the phrase "motive power equipment." It had clouded the real meaning of what might be termed "steam roadbed electrification." He always had been and would remain a strong advocate of direct-current in the interurban and short railroad field, the field where it belonged.

The actual power required to move a given tonnage when employing direct current was, he said, 22 per cent more than with single-phase. The question of motive power was an important one but a point not to be overlooked was the system behind that motive power. The locomotive as outlined in the paper might be a gold mine metaphorically speaking, but that was no reason for burning 22 per cent more coal to run it. As to single-phase railway installations, Mr. Armstrong had compared a.c.-d.c. trolley experiments without considering a single locomotive. He called attention to the New Haven Railroad which employed 100 locomotives, moved 65,000 cars each month, and used 65 multiple-unit cars in a 33-mile suburban zone. Yet Mr. Armstrong wanted his comparison to be accepted as proof that the single-phase system was not only passing but had actually passed. The Butte, Anaconda & Pacific Railroad was not an interurban property, but because of the short run, 22 miles, and the consequent impossibility of congestion, it was obvious that this low-speed railway ought to operate satisfactorily with high-tension direct current.

Single-Phase Systems

Again, he said, Mr. Armstrong's list of main-line electrifications was composed chiefly of short terminal and tunnel lines. No mention had been made of the great single-phase installations on the Spokane & Inland Empire Railroad nor on the New York, Westchester & Boston Railroad, for which steam would have been used had electricity not been at hand. No reference had been made to the wide adoption of single-phase systems on the continent of Europe, nor to the London & Brighton Railway in England, of which the directors had recently voted $10,000,000 in a recent appropriation to carry the electrification of the line through to Brighton. This was, he said, a good testimonial to the advantages of single-phase current. Mr. Murray referred to the communication from H. A. Gallwey recently published in the "Electric Railway Journal." This had announced a mileage of 55,000 for the locomotives on the Butte, Anaconda & Pacific Railroad during four months of operation, but during the same period the New Haven passenger locomotives alone had traveled 574,000 miles. The seventeen Butte locomotives average 22 miles a day, while some of the New Haven locomotives run ten times that distance daily.

Mr. Armstrong had said that the split-phase locomotive was 35 per cent heavier than an equally capable direct-current machine. Yet the New Haven had tested commercially a single-phase locomotive with continuous capacity of 1400 hp, the weight being 110 tons, while Mr. Armstrong had announced that the split-phase locomotive, its assumed successor, weighed 135 tons.

Mr. Murray then presented maintenance figures upon the cars of the New York, Westchester & Boston Railway. These, in cents per car mile, are as follows:

1912	July	2.6	1913	February	1.9
	August	3.5		March	2.5
	September ...	3.5		April	2.0
	October	3.4		May	1.2
	November	2.4		June	1.3
	December	2.7		September ...	1.3
1913	January	3.0		October	1.1

Since the improved New Haven locomotive repair shops had been built at Van Nest he stated that the running costs obtained for a.c.-d.c. locomotives were very low. After adding the general overhauling charges, the total cost of maintaining the passenger locomotives would be found to be between 5 cents and 6 cents per locomotive mile, and for freight engines of 110 tons, hauling 1500-ton trains and having 40,000 lb. maximum tractive effort, the total cost of maintenance was from 4 to 5 cents. For switching locomotives of equal tractive effort the cost was below 4 cents. Locomotives had given 25,000 miles of service to one detention.

Mr. Armstrong concluded the discussion by the statement that for the last five years the average maintenance cost of forty-seven direct-current locomotives on the New York Central & Hudson River Railroad had been 3.5 cents per locomotive mile.

Abuse of Water-Purification Plants

Plea for More Efficient Operation of Filters and Instances of Absurd Practices at Existing Works

That small water-purification plants receive, as a rule, little or no attention and that their effluents could be greatly improved under proper operation are the belief of Mr. H. P. Letton, sanitary engineer of the U. S. Public Health Service, Washington, D. C., expressed in a paper read at the meeting of the New England Waterworks Association Dec. 10. In most cases, when the purification plant is added to an existing system, the operation of it is intrusted to the engineer of the old plant. This man may be, and in many cases is, a stationary engineer who thoroughly understands the operation of boilers, engines and pumps, but who has absolutely no idea of the principles which underlie the process of water purification.

When it is decided that some form of purification is advisable in connection with a small water plant it is very rare indeed that a consulting engineer is called in for advice. Instead, the matter is taken up with one or more companies engaged in the business of installing purification apparatus. While these companies are usually competent to give reliable advice, their main business is the selling of equipment. Because of this fact and because the usual small water company or municipality is weak financially, the filter plant is designed to fit the available money, and the company offering the equipment for the least sum is generally given the contract regardless of the quality of the material to be furnished. As a result of this practice plants that are poorly designed, lacking in the necessary equipment for efficiency, or even wholly ill adapted to the situation, are commonly met with.

Example of Incompetent Operation

The worst example was at a plant supplying about 1,000,000 gal. per day. The supply was originally obtained from artesian wells, but on account of the high iron content of the water a new supply was obtained from an artificial lake. This water was highly colored by its passage through cedar swamps. From the lake the water flowed by gravity to the coagulation basin, a rectangular wooden tank of such size that normally less than twenty minutes were allowed for coagulation and sedimentation. During periods of high consumption the time was considerably decreased. The basin was set at such an elevation that at times of low water in the lake it was impossible to obtain the normal supply except by by-passing some raw water.

From the coagulation basin the water flowed by gravity to four rapid sand filters of the gravity, circular wooden-tank type, without loss of head gages or rate controllers. The agitating rakes were intended to be driven by a water motor, but the necessary power was lacking. The beds were therefore not agitated during washing. From the filters the water passed to a suction well which had been used in connection with the former well supply, and which provided less than thirty minutes' storage.

Two solution tanks had been provided and connected with a small displacement pump driven from the line shaft to which the main water pumps were attached. The small solution pump was intended to force a solution of sulphate of alumina into the raw water just before it entered the coagulation basin. At the time of Mr. Letton's first visit to this plant the chemical pump was out of order and no chemical was being added. It was afterward learned that this was its chronic condition. Tests of the raw water showed it to be slightly acid owing probably to humic acid from the cedar swamps. Consequently the addition of sulphate of alumina would be an absolute loss as far as results are concerned. Analyses of the raw and filtered water showed no material difference, and the filtered water had at times a color as high as 200.

Coagulating Difficulties

Two other plants were discovered treating acid water with sulphate of alumina only. Rate controllers and loss of head gages were almost unknown and the methods of regulating the amount of chemical applied were very crude. Few calibrated orifice boxes were found in use. Two gravity, rapid sand plants treating waters high in organic matter, and often turbid, had no coagulation basins, so that they required too frequent washing, with corresponding reductions in bacterial efficiency, entailing as well increased costs of operation.

Another large plant, using the pressure type of filter, was treating a turbid water with very little time for coagulation. The effluent was frequently turbid, and at times contained aluminum hydrate. One plant of the slow type had less than 1 ft. of filtering sand, and had a clean-water well holding about half an hour's supply. As a result of this combination the rate of filtration fluctuated exactly as the demand, and purification was practically nil.

At one rapid sand plant the clear-water basin was so small that it was necessary to wash with raw water. Another plant had no arrangement for filtering to waste, so that whenever it was necessary to get at the strainer system the dirty water in the bed was drained into the clear-water well.

"Rules of Thumb"

Men in charge of small water-purification plants usually have little or no idea of the nature of the process. They operate the plant by "rule-of-thumb" methods in an endeavor to produce a good-looking water. Although efficient results depend so largely on the use of the correct quantity of coagulant, the greatest ignorance was shown as far as this was concerned. At several plants the engineer stated that he put in a certain number of buckets of alum per day. He did not know how many pounds were used and made no attempt to add it in the same proportion at all times. A few engineers said that they increased the dose "some" when the water was turbid, but did not know how much. In only a few plants was any attempt made to regulate the dosage by the aid of alkalinity and turbidity tests.

Few of the men in charge could be depended upon to make the necessary alkalinity tests. At only two plants treating less than 20,000,000 gal. per day were laboratories maintained, and at these plants the tests, both chemical and bacteriological, were made under the direction of non-resident chemists. The engineers who made the tests were unable to interpret them or apply them in the operation of the plants.

At one modern municipal plant absolutely no records were kept. The chief engineer could neither read nor write and could see no use in records of any kind, not excepting pumpage records. The president of the board of water commissioners in charge of the plant stated that the only reason he could see for filtering water was to remove turbidity, nothwithstanding the fact that the sewage of more than 100,000 persons was discharged in the river from which the supply was taken, about 17 miles above the intake.

At one plant it was found that through the laziness of the engineer the filters were not being washed enough, the deficiency of water due to clogging being made up by bypassing raw water. At another plant the beds were washed too often, resulting in a low bacterial efficiency and a high cost of operation. More than 10 per cent of all water filtered was being used for wash water.

The Remedy

The most feasible remedy for poor design is a statute requiring the submission of plans for proposed plants or changes in existing plants to the State board of health, or some other State authority, for approval before construction can legally be carried out. Such a regulation is in effect in a number of States. Coupled with the appointment of a properly qualified engineer to pass upon all plans it will satisfactorily care for new works. State supervision for existing plants will, if carried out, do much to remedy the more serious imperfections of construction and operation. This supervision cannot, however, be thorough enough to furnish definite information for the efficient operation of the plant, both from an economic and a sanitary standpoint. At best the plant can be visited only about once a month, and this visit is merely to see that a safe and potable water is being produced. From the viewpoint of the State this result is sufficient, but from that of the consumer and taxpayer there are other matters of great importance. He wishes to know, first, that the water is healthful, and then that it is not possible to obtain the same or better results at a less cost by some change in the method of construction or operation of the plant. The latter consideration is out of the field of the State inspector, for it generally requires much experimenting and testing to obtain the point of maximum efficiency.

Daily Tests

The plan of having the engineer at the plant make daily tests under the general direction of a consultant at a distance is a step in advance, but it is far from perfect. The engineer's results are open to question, since he is frequently hurried and makes the tests without knowing the reason for the various steps. Also since he has but infrequent consultations with the consulting engineer or chemist and is unable in most cases to explain any seeming discrepancy in his results they do not prove to be of as much value as might be expected.

Mr. Letton has had in mind for some time a plan for remedying this situation, and it has recently been brought to his attention again by a similar plan which Prof. Earle B. Phelps has put into operation in connection with local health administration in several small Massachusetts towns. The plan is this: For several water companies or municipalities located not too far apart to combine forces, fit up a laboratory at some central point and employ a competent bacteriologist and chemist to give his entire time to the scientific supervision of their water plants. By so doing, each plant would have the benefit of expert advice at a small cost.

Illinois Central Improvement at Memphis

Track Elevation and Realignment in Conjunction with Construction of New Union Station—Extensive Use of Reinforced Concrete

THE Illinois Central and the Yazoo & Mississippi Valley railroads are building a new union station in the city of Memphis to replace two smaller stations used by both companies, and in conjunction with this the tracks in the vicinity are being raised a maximum of about 10 ft., a number of grade crossings are being eliminated and the alignment is undergoing revision to lessen the curvature and get the tracks out of Nettleton Avenue. Reinforced concrete has been used extensively both in the retaining walls and in some of the bridges, and where steelwork is unavoidable on account of sharp skews or lack of headroom for deep floors the steel is being incased in concrete. While the old alignment is being somewhat changed, the new location is so near the old that some difficulty is being encountered in maintaining traffic during the progress of the work.

At the present time the two companies, as previously stated, maintain two passenger stations. One of these, the Poplar Street station, is located north of the business district and on the waterfront; the other, known as the Calhoun Street station, is at the corner of Calhoun Avenue and Main Street, somewhat south of the principal business district. As the general plan indicates in part, the route from the Poplar Street station follows south in Tennessee Street to Huling Avenue. From this point the freight tracks continue in Tennessee Street to Calhoun Avenue, where they begin to swing to the eastward. The single passenger track, however, turns off sharply at Huling Avenue and swings into Nettleton Avenue, reversing back into an alley between Front and Main Streets. At Carolina Avenue it rejoins the freight tracks.

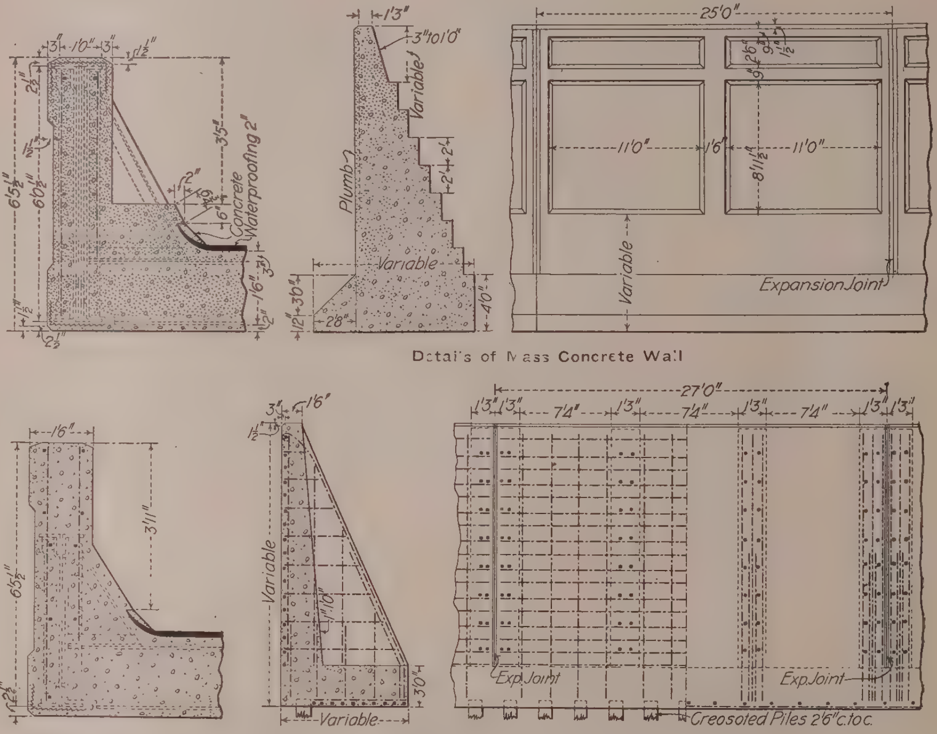
Outline of Improvements

Through Illinois Central trains now pass through both stations, and the Yazoo & Mississippi Valley trains to and from the south, terminating at Memphis, also reach both stations, the Poplar Street station being the terminal. The new station, which is practically on the site of the old Calhoun Street station, will be a combination through and stub station, and when it is put into service the Yazoo & Mississippi Valley trains will cease running through to Poplar Street. The Chicago, Rock Island & Pacific and the St. Louis & San Francisco will also use the station as a terminal.

The track elevation to eliminate the grade crossings begins at the Huling Avenue turnout. Swinging over Nettleton Avenue to a new right-of-way south of the avenue, it turns back to the south, crossing the present line of Butler Avenue, eliminating the curve at Calhoun Avenue, and giving a straight run through the station. The track elevation terminates at the railroad grade crossing in Railroad Avenue. It will be noted that the new location has two tracks throughout and three tracks most of the way.

The Nettleton Avenue crossing is more involved than any of the others on account of the proximity to the Huling Avenue junction, the sharp curvature (20 deg. for the inner track and 19 deg. for the outer), the sharp skew and the necessity for keeping Wagner Place open. The present tracks descend on a 2.4 per cent grade from Huling Avenue, and by using a 1 per cent ascending grade on the new work an elevation of about 12 ft. is obtained at the crossing. The bridge will be a three-span plate-girder and I-beam structure, with the north abutment on the curb line and the two intermediate bents in the center of the street and at the south curb line respectively. By moving the north abutment to the curb line the street, formerly 60 ft. wide, is narrowed to 52 ft., and one short span is eliminated. The I-beams span at right angles to the street except for the part over Wagner Place, where the 20-ft. opening is spanned direct from abutment to abutment with I-beams.

On account of the skew and curvature of this bridge the girder spans are of various lengths up to 55½ ft., and the girders are therefore of various depths. All will be incased in concrete built up to a uniform height, with ornamental panels. The floorbeams and bents will also be solidly incased in concrete.



Details of Mass Concrete Wall

Girder Protection

Details of Reinforced Concrete Wall

Details of Retaining Walls, and Concrete Covering of Bridge Girders

Details are shown of the incasing and the waterproofing.

This bridge as designed will have a floor depth of slightly more than 3 ft. This, with the required headroom of 12 ft., necessitates cutting Nettleton Avenue down 3 or 4 ft. In order to give Wagner Place under the bridge sufficient headroom, and yet leave it and the building of the Miller Hardware Company north of the avenue undisturbed, it was found expedient to divide the Nettleton Avenue roadway into two parts by a retaining wall. The

Progress View of Track Elevation under Traffic

Gravity Retaining Wall on West Side of Station roadway north of the wall will be left at its present level from Wagner Place east, while that on the south side will be cut down 3 ft. The north abutment will be extended as a low retaining wall to support the service track to the hardware company.

Numerous buildings on the south side of Nettleton Avenue were torn down when the new right-of-way was acquired. On account of these buildings it was impossible to run the location out on the ground, and therefore the layout was determined from a paper location on a large scale.

Butler Avenue will be spanned by an I-beam bridge incased in concrete. The opening is only 30 ft. wide, and the beams will span direct from abutment to abutment. The four stub-tracks now branching out at this point are considerably higher than the main tracks, being reached by a 5½ per cent ascending grade, and they will be readily taken care of with the new arrangement by changing this grade to a 4½ per cent descending grade.

CAROLINA AVENUE

Carolina Avenue will be crossed on a reinforced-concrete bridge of four spans. As the plan shows, the bridge will accommodate both passenger and freight tracks, ten in number, and will be about 210 ft. wide. The three intermediate bents, on the two curb lines and in the center of the roadway respectively, will be of uniform design, with 18-in. circular columns with spiral reinforcement, 6 ft. on centers, connected with continuous girders built to the form of circular arches. The bridge floor will be of thick concrete slabs protected with 2-in. waterproofing.

The bridge will have a floor depth of 4 ft. 3 in., and the underclearance line will be practically on the present ground line. The street will be cut down about 12 ft., with a 4 per cent approach grade at the west end and a lighter grade of 2.85 per cent at the east end. Main Street will also have to be cut down 8 ft. at its intersection with Carolina Avenue. It will be necessary to underpin the foundation of several large warehouses abutting the avenue, and an extra story will be built under one manufacturing plant. The existing sanitary sewer will be lowered about 4 ft. and moved from the center of the street to the north side to make room for the center bent. It will rejoin the old sewer at a manhole east of Main Street. A 24 and 30-in. storm sewer will be built on the west side of the roadway.

Numerous retaining walls will be required to hold the embankment on narrow right-of-way and to separate adjacent tracks at different elevations. Both gravity and reinforced walls are being used, the latter predominating for the reason that the gravity type is used only where there is room for a toe to take the overturning moment, and in many instance there is not such room. Typical sections and other details are shown of both the gravity and the reinforced types.

NEW STATION

As previously noted, and as the plan shows, the new station will be a combination of the through and stub types, with ten tracks in all. A double crossover at each end will make it possible to reach each station track from each main track. When the station is completed it is expected that the daily traffic will amount to about 60 trains in and out, 20 per cent of these being through trains and the remaining 80 per cent terminating at the station. The through trains will naturally be the longest. Some of the platforms accommodating them will be 940 ft. long, while the minimum length for those accommodating the stub tracks being 700 ft. long.

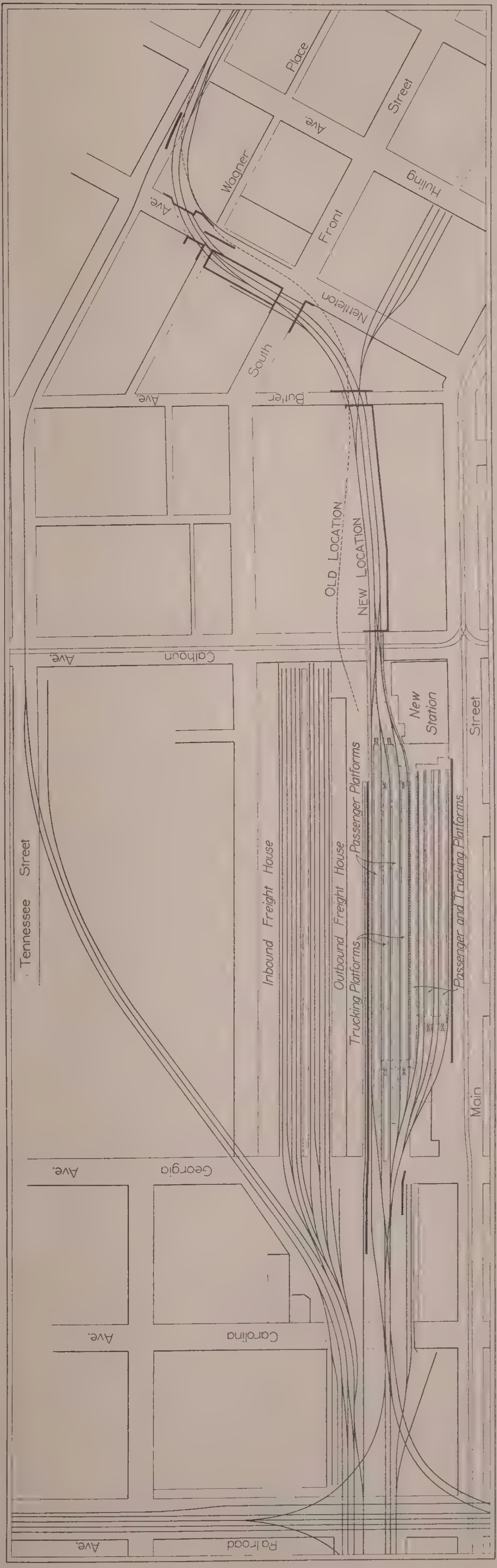
A notable feature of the track layout is that the through tracks are arranged singly instead of in pairs. This, with the alternating passenger and truck platforms, will keep baggage, mail and express traffic entirely off the passenger platforms. The passenger and trucking platforms are respectively 13 ft. 2 in. and 12 ft. wide.

The station building proper will be 250 x 150 ft., and will have eight stories, accommodating the Illinois Central and the Yazoo & Mississippi Valley offices. Care has been taken in the design of the station to break up into short flights the ascent from the street level to the tracks. Duplicate facilities will be provided for colored passengers, arranged in such a way that the two races will come together only at the train gates. Immigrant rooms will also be provided, and on the ground floor there will be an emergency hospital.

Architecturally the building will be of the Roman Doric type, the main motif consisting of a colonnade rising from the sidewalks up to the first cornice line, framed in by broad corner piers. The structure will be of Bedford stone for the first three stories, with a combination of brick and terra-cotta trimming for those above. The interior halls will have the same general character in design, being treated in ornamental plasters, with wainscoting effects of marble or enamelled brick. The building will have an office area of 4600 sq. ft. and will use about 1200 tons of steel. Work on the station has been started and the steelwork is being erected.

METHOD OF PROSECUTION.

By deferring the construction of the bridge at Butler Avenue, where the present line crosses the new, until the new line can be put into service traffic is being readily maintained from



General Plan of Memphis Improvement, Showing New Station, Intersecting Streets, and Relation of Old Location to New

Calhoun Avenue north. From Calhoun Avenue south the new station layout occupies in part the site of the old, and temporary provisions were necessary. A temporary station was built to the west of the old one, the foundations for the permanent structure to carry the tracks having first been laid so that it would be possible to place the columns to support the permanent tracks up through the temporary structure while keeping it in service. The long retaining wall on the west side was first built, and the placing of embankment was begun next to the wall. As the embankment is widened the tracks are laid on their final location, and are provided with temporary platforms and canopies, while tracks on the lower level are removed to make room for more fill. When the filling reaches the baggage tunnel under construction between the through and the stub portions of the station the embankment work will be held up until the tunnel is completed, but enough tracks on the upper level will be in service so as not to interfere with the operation of trains.

The Bates & Rogers Construction Company, of Chicago, has the contract for the grading and the masonry in connection with the track elevation. The American Bridge Company is furnishing the steel, with the exception of that for the Wagner Place and Nettleton Avenue subway, which is being supplied by the Louisville Bridge & Iron Company. The James Alexander Construction Company, of Memphis, has been awarded the contract for the construction of the station building, plans for which were drawn by D. H. Burnham & Company, of Chicago.

The track-elevation work will cost about $750,000, and the new station about $550,000. The work is being done under the direction of Mr. A. S. Baldwin, chief engineer; F. L. Thompson, engineer of construction, and Maro Johnson, engineer of bridges and buildings. Mr. W. G. Arn, assistant engineer, is in direct charge of the work.

Sealing Flashboards

A MORE or less temporary way of raising the water level back of a dam not outfitted with a movable crest is to put up flashboards and support them on steel pins. These pins can be so designed as to break or bend before a water level, critical for the safety of the dam, has been reached. At the Holtwood plant of the Pennsylvania Water & Power Company such an arrangement has been in use for several years, as described in the Engineering Record of Dec. 7, 1912, page 628.

The power company experienced at first some trouble in finding the right method of sealing these flashboards and has, therefore, experimented with various materials. The vertical joints are sealed fairly well by the water pressure itself, as the boards overlap each other 6 in., but the joint between the lower edge of the boards and the masonry has given the engineers some mental exercise. At first sawdust was tried, then straw, then cinders, and finally cinders and straw well mixed before application. This latter composition has proved satisfactory. In it the cinders form the body, while the straw is the elastic tightening medium. Only about 2 cu. yd. of straw are necessary to make the 2000 ft. joint practically watertight. When last summer all the water of the river was diverted through the power house during a short low water period a small measuring weir was constructed in the drainage channel of the river bed and the remarkable low leakage discharge of 1.7 sec. ft. was measured.

Strength of Outstanding Flanges in Beams and Columns

Relation between Strength and Dimensions of a Member and Effect of Varying Its Proportions

By R. J. ROARK
College of Engineering, Madison, Wisconsin

IN the determination of the economical section of beams and columns the fundamental problem is that of obtaining a maximum section modulus with a minimum amount of material. The attempt to do this has led to the adoption of sections comprised of relatively wide and thin parts, such forms concentrating the material farthest from the gravity axes. An example of this is seen in the special Bethlehem girders, with their wide flanges and thin webs. In a member of this sort, either a column or a beam, failure may occur through the local buckling of a flange or web plate before the full strength of the member as a whole has been developed.

An exact analysis of the stresses in a flange or web of this kind would be extremely difficult, if not impossible. It would combine the difficulties of column and flat plate investigation, and in the present state of knowledge may be regarded as impracticable. By making certain assumptions, however, some relation between the strength and dimensions of such a member can be determined, which, while not accurate, may serve to show in a general way the effect of varying proportions. An attempt will be made to determine such a relation for the case of an outstanding flange.

Let l = the length of the member in inches; b = width of flange; t = thickness of flange; p = unit load on the flange in pounds per square inch; y = the lateral deflection of any point along the flange from its original position; and E = modulus of elasticity of the material in pounds per square inch.

The flange is shown in Fig. 1. It is assumed to be rigidly held along the base AB by the rest of the member.

Any elementary transverse strip of the flange, as ab, may be regarded as a cantilever beam. Any elementary longitudinal strip, as cd, may be regarded as a column. The load on each of these elementary columns causes a tendency to bend, and this tendency is resisted partly by the column itself and partly by the reactions, r, of the elementary cantilevers against the column. Each cantilever is loaded by the lateral thrust of the elementary columns, and since the thrust of any column is proportional to its middle deflection, each cantilever is loaded at any point according to its deflection at that point. This gives a condition of loading for the elementary cantilever intermediate between that of a uniform and that of a concentrated load. Since the error involved is slight, the elastic curve of each elementary cantilever beam will be assumed to be a parabola. The difference between the sinusoid given by Euler's theory and a parabola is slight for small curvatures and the elastic curve of each elementary column will also be assumed to be a parabola.

Since the elastic resistance of a long column is independent of its deflection, each of the elementary columns in the flange will carry the same load. Hence the entire flange may be taken as a single column, and the load carried by it directly, as given by Euler's formula, is

$$P = \pi^2 E t^3 b/12 l^2$$

The remaining load, which will be denoted by P', is equal to

$$P' = p\, t\, b - \pi^2 E\, t^3 b/12 l^2 \qquad (1)$$

This remaining load is resisted by the cantilever action of the flange, and hence causes a certain bending moment along AB. It is desired to find this bending moment.

FINDING BENDING MOMENT

Let any elementary column, as cd, be considered. As in the case of the entire flange, a certain portion of the load coming on this elementary column is resisted by column action. The remainder of the load, which will be denoted by p', is resisted by the combined reactions r of the elementary cantilevers. If all these reactions were concentrated at o, the middle of the column, the amount required to balance p' could be found as follows: Taking moments about o

$$2\, p' y = \Sigma r\, l/2$$

hence

$$\Sigma r = 4\, p' y/l$$

Again, if the reactions acted uniformly along the length of cd, the amount required would, from analogy with the case of a simple beam, be just twice as great, and we would have

$$\Sigma r = 8\, p' y/l$$

Since the reactions are neither concentrated nor uniformly distributed, but vary according to the deflection at any point, an intermediate value for Σr may be taken. It will be assumed that

$$\Sigma r = 6\, p' y/l \qquad (2)$$

The moment produced at AB by each elementary column is equal to the product of Σr and the distance from the column in question to AB. The total moment produced at AB is equal to the summation of these products across the breadth of the flange. This is the same as the product of the total thrust and the distance from AB at which it acts. From the assumption that the curve of the elementary cantilever is a parabola it follows that the average value of y is $1/3\, y'$. Hence, substituting in (2) the value of $1/3\, y'$ for y, and the total load P' for p', we determine the total thrust T, tending to cause a moment at AB to be

$$T = 6\, P' y'/3l$$

This thrust is made up of a series of thrusts which are proportional to the values of y, and hence it acts at the center of gravity of the area $x\, o\, y$. From the properties of the parabola it follows that this center of gravity is distant $3/4\, b$ from AB. The bending moment at AB is therefore

$$M_{AB} = \frac{6\, P' \frac{1}{3} y'}{l} \times \frac{3}{4} b$$

$$= \frac{3}{2} \frac{P' y' b}{l} \qquad (3)$$

The resisting moment along AB will now be found in terms of the various dimensions.

The resisting moment of any elementary cantilever beam as *ab* is equal to

$$M_r = 3.5\, y\, E\, I/b^2$$

The coefficient 3.5 is taken as intermediate between the coefficient 3 for a cantilever loaded at the end, and the coefficient 4 for a cantilever uniformly loaded.

At any point along the flange edge, the curve being a parabola, we have

$$y = \left(1 - \frac{4\,z^2}{l^2}\right) y'$$

Hence the resisting moment of each cantilever beam is equal to

$$M'_r = y' \left(1 - \frac{4\,z^2}{l^2}\right)\left(\frac{3.5\, E\, I}{b^2}\right)$$

Substituting for I its value $1/12\, t^3\, d\, z$ we have as the total resisting moment in the length, l

$$M'_r = \frac{2\,E\,t^3}{3.43\, b^2} \int_0^{\frac{l}{2}} y' \left(1 - \frac{4\,z^2}{l^2}\right) dz = \frac{E\, t^3\, l\, y'}{5.15\, b^2} \tag{4}$$

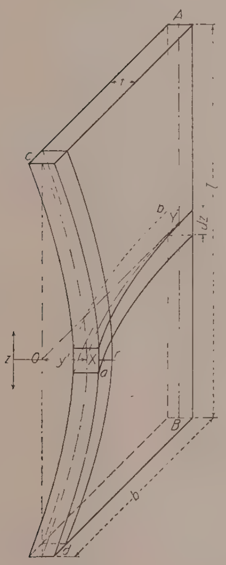


Fig. 1

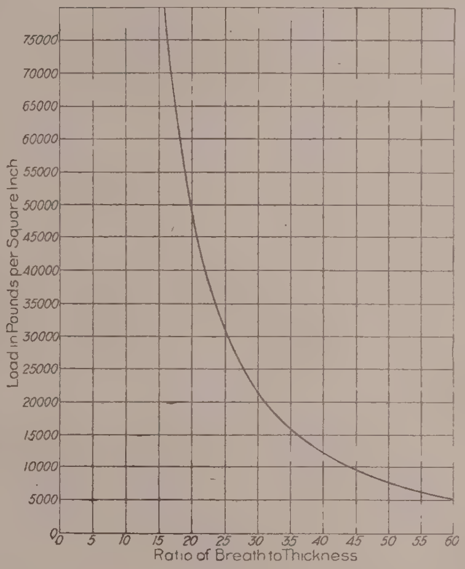


Fig. 2

Equating the bending moment (3) and the resisting moment (4) we have

$$3/2 \times P'\, y'\, b/l = E\, t^3\, l\, y'/5.15\, b^2$$

Substituting for P' its value in (1) we get

$$\frac{3}{2}\left(p\, t\, b - \frac{\pi^2\, E\, t^3\, b}{12\, l^2}\right)\frac{b\, y'}{l} = \frac{E\, t^3\, l\, y'}{5.15\, b^2}$$

Solving this for p gives

$$p = E\, t^2\, l^2/7.7\, b^4 + \pi^2\, E\, t^2/12\, l^2$$

It will be noted that p depends upon two functions, one of which, the resistance due to cantilever action, varies directly as l^2, and the other of which, the resistance due to column action, varies inversely as l^2. Now l is not necessarily the total length of the flange, but only the length over which flexure in one direction extends. This length may be anything less than the total length of the flange, and so that length giving the least value of p should be taken. Differentiating the above expression, equating to 0 and solving for l, we get $l = 1.59\, b$, say $1.6\, b$, which by trial is found to give a minimum. Substituting this value for l, and reducing, we get, as the final expression for the strength of an outstanding flange,

$$p = 0.65\, E\, t^2/b^2$$

The curve of this equation is shown in Fig. 2.

This expression is similar to Euler's formula for columns in that it gives the elastic resistance to buckling only, and takes no account of the compressive strength of the material. Since the flange need only be strong enough to take the same unit load as the column as a whole is designed to take, the ultimate compressive strength need not be considered. For any unit load the formula gives the maximum ratio width of flange to thickness. If the full compressive strength of the metal is to be developed, the unit load on the flange is, say, 36,000 lb. per square inch, and we have

$$36{,}000 = 0.65 \times 30{,}000{,}00\, t^2/b^2$$

whence

$$t/b = .043, \text{ or } b = 23\, t.$$

Buckling Strength

So far as the writer knows, no experiments have been made for the purpose of determining the buckling strength of flanges. In some tests on columns and beams, failure has occurred through such buckling, and this fact has led some to believe that the flanges so failing were too wide and thin.

In tests at the Watertown Arsenal on rolled H-sections several specimens failed by buckling of the flanges. The detailed dimensions of these sections are not reported, but the flanges were about 3 in. wide and 0.39 in. thick. These specimens failed at a load of about 45,000 lb. per square inch. According to the above formula the flanges should have had a strength against buckling greater than this, but as the elastic limit was exceeded the formula could not be regarded as applicable to conditions of failure. There were several built I-columns which also failed by buckling of the flanges. These columns had flanges 4 in. wide and ⅜ in. thick. Failure occurred at about 36,000 lb. per square inch, which was slightly above the elastic limit of the material. According to the formula derived above, these flanges had a strength against buckling of 0.65 × 30,000,000 × 0.141/16 = 171,000 lb. per square inch, indicating that failure would not take place below the elastic limit.

In a series of tests made by Professor Marburg on Bethlehem girders with wide flanges several specimens failed by buckling of the compression flange. These girders had flanges 1 in. thick next the web and 10.5 in. wide. The modulus of rupture as computed from the load at failure was 54,000 lb. per square inch. According to the formula, the flanges of these girders were capable of taking a load of 0.65 × 30,000,000 × 1/25 = 780,000 lb. per square inch, as far as elastic deflection was concerned.

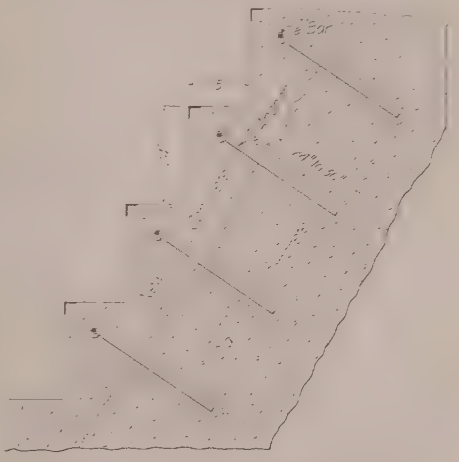
In all the above cases failure occurred after the elastic limit was reached. According to the above formula, the flanges were strong enough to insure against buckling under loads below the elastic limit, and this proved to be the case. The fact that ultimate failure took place by wrinkling of the flanges should not be necessarily regarded as a proof that they were excessively wide.

In order to test the formula given below, it would be necessary to make strain measurements on flanges of varying proportions. By noting the maximum load supported, or the load at which deflection began, a constant might be determined which would give more accurate results than the coefficient 0.65.

As the formula stands, it would appear that much wider and thinner flanges could be used than any of the standard or special sections now made have. It should be remembered, however, that the analysis given above assumes that the flange is held with absolute rigidity along the supported edge. In an actual member this would not usually be the case, as the tendency of the flange to bend would induce torsional stresses in the member. Just how much difference this would make could only be determined by experiment.

Patching Disintegrated Concrete

Disintegrated concrete is now being chipped out and replaced on Drydock 2 at the New York Navy Yard. The reasons for this disintegration have not as yet been determined, but it seems to have been caused either by the influence of salt water or the action of frost.



Method Used in Patching Concrete

It finally became necessary to repair the walls and the floor of the dock. The concrete faces were therefore chipped off until layers of good concrete were reached. From this face holes were drilled normal to the wall to a depth of from 24 to 36 in. and anchors made of bent ⅞-in. square, twisted bars grouted into them. To the eyes of these anchors longitudinal ⅞-in. twisted bars were attached, so that when the concrete is deposited it is actually "pinned" to the existing wall.

The accompanying sketch illustrates this method. The concrete used is composed of 1 part cement, 3 of sand and 6 of gravel. On the floor the same method has been used, except that the longitudinal bars are eliminated and panels of expanded metal used instead. These panels are prevented from bulging upward, if water pressure should be acting from below, by twisted bars crossed over them.

The Effect of Frost on the lead service pipes of the city of Liverpool, England, is mentioned in the 1912 annual report of Mr. Joseph Parry, chief engineer of waterworks. Between Jan. 28 and Feb. 5 there were reported 10,948 burst lead pipes and 61 burst iron pipes. The consumption of water in and around the city rose from 205,000,000 gal. for the week before the frost to 272,000,000 gal.—an increase of 32 per cent.

Contractor, Engineer and Inspector

Discussion of F. L. Cranford's Paper before the Road Builders' Association at Philadelphia, Dec. 10, by Fred E. Ellis, Manager Essex Trap Rock & Construction Company, Peabody, Mass.

I WILL try to show what helps to make the relations between the engineer and contractor harmonious and what makes discord, by describing the workings of a contractor's mind while bidding on a contract and during construction.

The first knowledge that a contractor has that a road is to be let by contract is when he sees the advertisement inviting bids. For the purpose of illustration let us take a case where the work is in a strange territory and the contractor and engineer are unknown to each other even by reputation. The first feeling a contractor experiences is one of curiosity. He wonders what kind of a country it is, what kind of people live along the road, what kind of roads they own, what kind they propose to own, what kind of engineers they have and what kind of contractors. The spirit of competition is next aroused. The contractor feels that he would like to put his knowledge, experience and organization up against those of other contractors, and if given a good set of rules and a fair referee he has no fear of the result. He feels that he can do the work as well as, if not better than, his competitors, but he knows in order to get the work he must do it at less cost. He procures a set of plans and specifications, which are the rules under which the game is to be played, and since the principal thing which now interests him is the type of road which is to be built, he turns to that part which gives the description. If he finds that the road is of a kind with which he has had experience, he proceeds with greater confidence. He will find in the specifications nothing which shows any confidence in the contractor on the part of either the commission or the engineer; neither is there anything which inspires the contractor with any faith in the engineer.

MEASURING THE ENGINEER

He next views the location of the work. If he finds that the specifications fit the location, that there are evidences of individual study, that the engineer has taken advantage of the local conditions, and he is satisfied that he can do a good job under the specifications, the contractor begins to have confidence in the engineer. If, however, he finds that the specifications show little proof of individual study or there are evidences of incomplete surveys taken at unseasonable times of the year when accuracy is impossible, then he has not found ground in which to plant even the seed of confidence.

The next step is to get acquainted with the engineer—to have a chance to size him up, to give the engineer a chance to size the contractor up, and to find out in a general way how near to the specifications the engineer expects the work to be done, the specifications being drawn so accurately and with so little elasticity that it is impossible ever to build a road to the letter. It is absolutely necessary that the minds of engineer and contractor meet before any intelligent business-like bid can be made. Any bid made otherwise is in the nature of a gamble and not a business proposition. In order that there be no misunderstanding by what I mean by the work not being done in accordance with the letter of the specifications, I will give this illustration: Nearly all specifications contain a clause which reads in substance as follows: "Subgrade shall be true to lines and grades as shown upon the plan." I do not believe a road was ever built where the subgrade was absolutely true to the lines and grades as shown on plans. I am willing to confess that if this clause was interpreted literally I would still be working upon the subgrade of the first State road I ever contracted for, seventeen years ago. The difference in expense to the contractor in trying to obtain absolute accuracy and of doing work approximately, within the limits of good workmanship, is very great and when carried on throughout all the items in the contract means either a profit or a loss.

THE "FUSSY" ENGINEER

It is on such items as this that he "sizes up" the engineer in a general way and makes up his mind whether he belongs to the "big" class or the "fussy" class; and when this has been done he is in condition to make an intelligent bid—one which is based not upon the letter but upon the spirit of the specifications, as determined when the minds of the engineer and the contractor met. I might add that the bid made up with the so-called "fussy" engineer in charge will be somewhat higher than that with the liberal engineer.

The contractor likes to leave the engineer confident that he understands exactly what is required and that he can fulfill the requirements. He likes to feel that the engineer has the confidence of his superiors and that what he says goes with them; that the road is of the engineer's own design and is to be built under his directions, without any influence being brought to bear either from within or without the department. He likes to feel that the engineer believes that if the contract is awarded to him it will be carried out in accordance with the spirit of the specifications in such a manner that it will be a credit both to the engineer and the contractor. Having received the necessary information he makes out and submits a bid.

THE INSPECTOR

The next steps are awarding and signing the contract and beginning the work. If the work is done under the personal charge of the engineer it will go along smoothly, but if not, there appears upon the scene a third party in the person of the inspector, who is mentioned in the specifications under the clause "engineer or others upon whom authority is conferred." If the inspector has worked for a long time with the engineer, thinks the same and acts the same as the engineer, there is still no reason to doubt that the work will run along smoothly; in other words, the confidence which existed between the engineer and contractor will exist between the inspector and contractor. If, on the other hand, the inspector is unknown to the engineer and does not understand his mind and his methods, friction cannot be avoided.

The contractor has entered upon his contract in good faith and may be carrying out his contract in accordance with the spirit of the specifications; yet the work may be interrupted by the inspector. When such an interruption occurs the contractor receives a shock similar to that which a surgeon would receive if he were stopped in the midst of an operation by an inexperienced attendant whose only knowledge and authority is contained in an inaccurate written description of the operation as performed on some other person by another surgeon. He knows the patient will suffer as well as himself. Up to this time both contractor and engineer have lived up to the spirit of their agreement, each having mutual confidence in each other. The entrance of the inspector with views different from the engineer's places the contractor in an entirely different position from that which he occupied when the bid was made up.

The inspector is not alone responsible for this condition of affairs. We all know that the public clamor and demand for improved roads are far in advance of the spread of practical knowledge of good-roads construction. Most of the inspectors are recruited from the ranks of students who have been studying exact sciences or from persons whose only knowledge of road construction is contained in the book of specifications. Their knowledge of contractors in general is limited to what they have read in the newspapers—all being wealthy and dishonest, having no pride in their work and no object in doing it except to get away with all the money possible. The contract and specifications aid the inspector in taking this view. No one has told him that the greatest ambition of a road contractor is to be known in the community where he has done work as the man who built a good road—not as the man who made a pile of money and built a poor road.

INTERPRETING SPECIFICATIONS

It is plain to see that the cause for discord among contractor, engineer and inspector is in the different interpretation of the specifications; therefore, change the specifications. Have them state distinctly what will be required of the contractor and no more. Change the clauses which place the standard of accuracy so high that it is impossible to attain it. Arrange the specifications so that within the limit of good workmanship the contractor has a right to his opinion equal to that of the engineer. Give the contractor the right to take a claim to court unprejudiced by any opinion previously given by the engineer. Have courses of instruction for inspectors. Have contractors qualify as to experience and ability to carry out work upon which they are bidding. I believe that the results of these changes would be much better workmanship at less cost and far pleasanter relations among contractor, engineer and inspector.

ONE OF THE BEST AND MOST COMPREHENSIVE GUIDES for the estimates of percentages of the total cost of work which are due to the preparation of plans and subsequent supervision has been afforded by the latest report of the Public Works Department of New South Wales, Australia. There practically everything that can be called a public work is carried out by the State Government, which has a staff of experienced engineers. During the period treated of, small contracts—of which there were 1500 in number—and day labor were the prevailing methods employed, and the total amount expended was about $20,000,000. This covered the construction of railways and electric tramways, which formed the largest items, highways, bridges, irrigation, harbors, water conservation and supply, sewerage, ferries and public buildings over the entire State, of which the area is 310,000 sq. miles, necessarily involving much traveling expense. The cost of administration, surveys, design and supervision amounted to 6 per cent of the cost of the actual work.

Economical Development of a Water Power

Description of a Method, Including Notes on How to Arrive at the Economical Diameter of a Penstock

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In the process of investigating the economies of the development of several water powers in the last few years the whole problem has invariably centered about the question of maximum wheel development which is economically justifiable.

The subject appears to have been dismissed in many cases as one of opinion or of custom, except as intimated in the paragraph below by C. T. Main in his paper before the American Society of Mechanical Engineers.

"It is customary to develop water powers to the sixth and seventh months in order of dryness of an average year, or more in special cases" (New England Waterworks "Journal," March, 1910, page 12).

Mr. Charles T. Main in a paper before the New England Waterworks Convention of March, 1912, stated: "In a large number of water-power developments which I have examined a very large percentage has been developed with wheel capacity sufficient to use all of the water from six to seven months in an average year, and during the remaining months water would go to waste. The economical development has been stated by some engineers to be nine months. No general statement is applicable to all conditions. It is a question of economies and requires solving for each particular case."

Mr. John R. Freeman, consulting engineer, assumes an economical development up to the yield of the eighth month. (Report on New York's Water Supply, page 422.)

While it is obvious that all engineering matters—this one, perhaps, in particular—should be passed upon by intelligent, experienced and educated judgment, the following is presented to show that the question, after all, is as Mr. C. T. Main states, "one of economies" and is capable of approximate solution for each particular case.

Average Monthly Runoff for Ten Years

The watershed area selected for the application of the following line of reasoning is one situated in the mountainous portion of central New England, where the monthly runoff for ten years, arranged in order of dryness, is as follows:

Month	Sec. ft. per sq. mile
Dryest	0.26
Second	0.44
Third	0.88
Fourth	1.07
Fifth	1.20
Sixth	1.36
Seventh	1.64
Eighth	2.01
Ninth	2.48
Tenth	3.08
Eleventh	3.82
Wettest	6.34

giving an average of 2.13 sec.-ft. per square mile

The first step in the process is to arrive at the economical diameter of penstock required. This may be accomplished in one of many ways among which is that of A. L. Adams, printed in the "Transactions" of the American Society of Civil Engineers, Vol. 59, page 177; or, perhaps, by using the following method, which is patterned after one already published by the author of this paper (see Engineering Record of Dec. 16, 1911, page 722), the question may be solved more quickly:

The average cost of penstock for this work per linear foot had been estimated as follows:

Diameter, in.	Cost per lin. ft. without excavation
60	$6.15
72	7.70
78	8.50
84	9.60
96	11.45

For the 84-in. diameter penstock it was found that there was an average earth excavation of 2.88 cu. yd., or 78 cu. ft. per linear foot of trench, which was equivalent to about $(d+2)^2$.

Assuming the cost of excavation to vary as $(d+2)^2$, the cost of excavation per linear foot is as follows, when a unit price of 65 cents per cubic yard is assumed:

Diameter, in.	Cost per lin. ft. of trench
60	$1.13
72	1.48
78	1.66
84	1.87
96	2.30

Or the total cost of a penstock per linear foot of pipe is as follows, including cost of excavation:

Diameter, in.	Pipe only per lin. ft.	Excavation per lin. ft.	Total	$X^{1.25}$
60	$6.15	$1.13	$7.28	$7.48
72	7.70	1.48	9.18	9.35
78	8.50	1.66	10.16	10.40
84	9.60	1.87	11.47	11.35
96	11.45	2.30	13.75	13.50

In the last column it is seen that the cost per linear foot in dollars may be given as the 5/4 power of the diameter in feet.

Assuming that the power is worth $0.01 per horsepower-hour, the value of the horsepower lost by friction is

$$\text{Val. } H_f \text{ hp} = (Q \times H_f/12) \times 30 \times 12 \times 0.01 \times 24 = 7.2\, H_f Q$$

The total value of the pipe and the capitalized value of the horsepower lost through friction then is

$$C = x^{1.25} + (H_f Q \times 365 \times 24 \times 0.01 \times yz)/(0.07 \times 12)$$

where

C = cost per linear foot plus capitalized cost of H_f losses, assuming capitalization at 7 per cent; Q, second-feet; y, ratio of the average friction losses actually occurring month by month to what the loss would have been had the full "maximum" capacity of the pipe been flowing during all the months; z, the ratio of average second-feet actually flowing to the second-feet assumed as the maximum development.

$$v = c\sqrt{ri};\ Q = ac\sqrt{ri} = D^2 \times 0.7854 \times 130\sqrt{ri}$$

$$Q = (D^{5/2} \times 0.7854 \times 130\sqrt{i})/2 = 51\, D^{5/2}\sqrt{i}$$

$$\frac{Q^2}{51^2 D^5} = 0.000385 \frac{Q^2}{D^5}$$

$$C = x^{1.25} + (H_f \times Q \times 365 \times 24 \times 0.01 \times yz)/(0.07 \times 12) = x^{1.25} + (7.29\, H_f Q yz)/0.07 = x^{1.25} + 104.3 H_f Q yz = x^{1.25} + 0.0402\, Q^3 yz/x^5$$

differentiating and placing the first derivative equal to zero

$$\frac{dc}{dx} = 1.25\, x^{0.25} - (5 \times 0.0402\, Q^3 y x^4 z)/x^{10} = 1.25\, x^{0.25} - 0.201\, Q^3 yz;$$

placing

$$\frac{dc}{dx} = 0$$

$$x = (0.161\, Q^3 yz)^{1/6.25} = 0.747\, Q^{0.481} (zy)^{0.16},$$

which is the Darcy formula, or using Hazen and Williams' formula,

$x = 0.786 Q^{0.465} (zy)^{0.1635}$ for twenty-four-hour power.

Similarly, it may be shown that for a twelve-hour power $x = 0.702 Q^{0.485} (zy)^{0.1635}$; and it is seen by inspection that the economical diameter for a twelve-hour power is 1.23 times the diameter for a twenty-four-hour power.

Approximate Formula

The economical diameter of a pipe for any purpose—where either the cost of pumping the water 1 ft. high or the value of the water falling 1 ft. is at the rate of 5 cents per 1,000,000 gal. raised 1 ft., equivalent to 0.87 cent per horsepower-hour, and for 73 per cent efficiency on water power—is expressed in inches: $13.5\sqrt[2\ 3]{Q''}$ (approximately) (see diagram 1), where Q'' is 1,000,000 gal. daily. This is equivalent to a velocity of 2.35 ft. per second. This may be stated more accurately by the following equation, also giving economical diameter in inches as $13.5\,(e/5)^{0.138}\,(Q'')^{1/2.3}$, where $e \Rightarrow$ cost to raise 1,000,000 gal. 1 ft. and Q'' = 1,000,000 gal. daily.

Both these formulæ were obtained by the inspection of a diagram published with an article by the author of this paper in the Engineering Record of Dec. 16, 1911, page 723. The formula by which the diagram was constructed is as follows:

$$2x^{6.87} + 1.4x^{5.87} = 8.3 E b m Q^{2.85}$$

or

$$Q = [(2x^{6.87} + 1.4x^{5.87})^{1/2.85}]/8.3 E b m$$

where x = required diameter of pipe in ft.; Q, second-feet; E, $20a$ = capitalized cost of raising 1 sec.-ft. 1 ft. per year; b, proportional part of time water flows in pipe; m, friction factor Hazen and Williams' formula = 0.000925 (old pipe), or 0.000568 (new pipe).

Using the approximate formula (which, by the way, is easily remembered) for the seventh month, for instance: $Q = 216.4$ sec.-ft. = 140 million gallons daily; the economic diameter = 115 in., and correcting for value of yz for this month $= (0.518)^{0.1635} = 0.899$, it becomes $D'' = 103$ in. Correcting again for one-half time (twelve hours being one-half of twenty-four hours) $D''' = D''/1.12$, or $D = 92$ in. (for 0.87 cent per horsepower-hour), and, correspondingly, $D = 94$ in. (for 1 cent per horsepower-hour capitalized at 7 per cent; $C = 130$; wood-stave pipe cost one-third to four-tenths that of cast-iron pipe. Some of the factors which tend to neutralize one another have been omitted in this solution. (See solution of this same problem in addendum.) By accurate formula $J = 92$ in.

Thus, for approximation, keeping in mind the rough corrections for certain factors, the approximate formula is fairly close, and at least serves to guide the judgment before making computations. For cast-iron pipe costs and $C = 100$, at a value of 5 cents per 1,000,000 gal. raised 1 ft., capitalized at 5 per cent, the formula is almost exact.

By means of a sample computation the method of arriving at the economical diameter will be demonstrated, but the results will be given for the eighth month only.

Sec.-ft.	H_f 78 in., diameter assumed	
17.2	0.012	
29.0	0.032	
58.1	0.117	
70.8	0.168	
79.2	0.208	
89.8	0.260	Development up to yield of eighth month in order of dryness.
133.0	0.370	
133.0	0.540	
133.0	0.540	
133.0	0.540	
133.0	0.540	
133.0	0.540	
1117.3	Average 0.322	

Average = 93.2 sec.-ft.; $z = 0.701$; $y = 0.596$; $x = 0.702Q^{0.485}(zy)^{0.1636}$ (twelve-hour power); $= 0.702\,(266)^{0.485}\,(0.701 \times 0.596)^{0.1636}$; $= 0.702 \times 13.45 \times 0.867 = 8.18$; therefore the economical diameter is 98 in. on a twelve-hour basis. For twenty-four-hour power with $Q' = \frac{1}{2}Q$ divide by 1.23, giving an economical diameter of 80 in.

Development to Eighth Month

Twelve-hour power; economical diameter, 98 in.; maximum discharge, 266 sec.-ft.; average discharge, 186.4 sec.-ft.; average H_f = (0.34); maximum H_f = (0.665); 305 — 3.88 = 301.12 and 305 — 7.52 = 297.48.

Horsepower maximum = (297.48 × 216.4) ÷ 12 = 6600 hp.

Horsepower average = (301.12 × 163.4) ÷ 12 = 4700 hp.

ESTIMATED COST OF PLANT

Dam	$21,000
Penstock	170,000
Standpipe	4,000
Equipment	211,000
	$406,000
Adding 10 per cent	40,600
	$446,600
Estimated value—land and water rights	200,000
	$646,600
Gross income 4,700 × $43.70	$206,000
Operating expenses, 21 per cent on $646,600	135,500
	$70,500

which is 10.9 per cent return on investment

Similarly, the computations were made for the other months, beginning with the fourth, in order of dryness, up to and including the ninth month, and the results are as follows:

Development to yield of number of month in order of dryness	Sec.-ft. per square mile	Max. sec.-ft.	Average sec.-ft.	Max. hp	Average hp	z	y	$(yz.^{0.1636})$	Economical diameter, ft.	Economical diameter, nearest even in.	Average return on investment, per cent
4	1.07	141.6	123.6	3520	3080	0.873	0.830	0.944	6.68	80	6.21
5	1.20	158.4	135.6	3960	3390	0.855	0.798	0.940	6.96	84	7.34
6	1.36	179.6	147.2	4470	3680	0.820	0.755	0.923	7.23	86	8.62
7	1.64	216.4	163.4	5400	4090	0.763	0.680	0.899	7.68	92	9.25
8	2.01	266.0	186.4	6600	4700	0.701	0.596	0.867	8.18	98	10.90*
9	2.48	328.0	207.0	8110	5170	0.632	0.513	0.832	8.62	104	10.60

*The maximum return on the investment apparently accrues from a development up to the eighth month, hence the eighth month here is the month to which the power should be developed.

z = the ratio of the average Q to the maximum Q for the month of development, and y = the ratio of the average H_f to the maximum for Q of the month of development.

For a watershed whose runoff is materially different from the above values, y and z must, of course, be determined anew.

It is obvious that some of the assumptions must of necessity be extremely approximate. For instance, the cost of equipment will not vary directly as the number of horsepower or kilowatt-hours required. Likewise the operating expenses will not vary directly as the investment. Furthermore, it is difficult to assign a reasonable value to the output even when all the conditions of the local market for light and power are known. Indeed, all these somewhat reasonable objections do tend to disparage the rationality of the results, in spite of which, however, the results themselves are upheld by the opinions and conventional practice, as quoted at the outset of the paper.

Addendum

The following is a solution of the problem of economical diameter of penstock by means of approximate formula and diagram:

Assume 216.4 sec.-ft. — seventh month in order of dryness. $C = 130$; value, 1 cent per horsepower-hour capitalized at 7 per cent; cost of penstock of wood-stave pipe, roughly, say four-tenths of the cost of cast-iron pipe of same diameter; twelve-hour power; $y = 0.576$ and $z = 0.701$.

Entering the diagram with 216.4 sec.-ft., the first determination of diameter is 115 in., running down the 115-in. line to its intersection with a diagonal 1-cent value of 1 hp-hour and reading on the scale in the center $D'' = 117$ in. Then, running on the horizontal line $D'' = 117$ in. to the diagonal $C = 130$, we read $D'' = 109$ in. on the horizontal scale. Running up the 109-in. line to cost of pipe equals four-tenths the cost of cast-iron pipe, we read $D'' = 124$ in. on the scale marked final diameter.

By our rule the economical diameter of penstock for a twelve-hour flow is 1/1.12 the diameter of one for a twenty-four-hour power, using the same Q. Hence we divide 124 in. by 1.12 and have a result of $D'' = 111$ in.

Corrections

This must further be corrected for losses due to the fact that the full quantity of water will be flowing in the pipe for five months in the year on the average, while for the other seven months the (H_f) friction losses will be very materially reduced, so that the average H_f loss for the year will amount to only 59.6 per cent of what it would have been had the full quantity been flowing the entire twelve months. Hence we multiply by $(y)^{0.164}$. Likewise a correction must be made for quantity Q by multiplying the average Q by $(z)^{0.164} = (0.701)^{0.164}$, making $(yz)^{0.164} = 0.899$.

Multiplying 111 in. by 899 and correcting for a capitalization of 7 per cent instead of 5 per cent, we arrive at an economical diameter of 94 in., or practically the same result obtained from the accurate formula, which gave 92 in.

Approximate formula $D'' = 13.5\,{}^{2}\sqrt[3]{Q}$ (apply corrections). Exact formula $D = 0.702Q^{0.485}(zy)^{0.164}$, where D = diameter in feet; Q, quantity in second-feet, and $C = 130$.

Value of 1 hp-hour equals the cost to raise 1,000,000 gal. 1 ft. high, or 5.75 cents capitalized at 7 per cent. The so-called approximate formula may be reduced very nearly to the terms of the so-called exact formula by applying the necessary corrections.

To show how much time may be saved by using the diagram of the approximate formula and applying the—in this case—few corrections, an example is given of another kind of problem.

The following problem was first solved by using the laborious, though extremely simple, and easily understood methods required to follow the so-called Adams method, which is:

"That pipe fulfils the requirements of greatest economy wherein the value of the energy annually lost in frictional resistance equals 0.4 of the annual cost of the pipe line." A. L. Adams in "Transactions" of the American Society of Civil Engineers, Vol. 59, page 177; also, Frye's "Civil Engineers' Pocketbook," page 1332.

Thus, if L = value of energy lost in frictional resistance and C = annual cost of pipe line, then $L = 0.4\,C$ for economical design.

This formula must be solved tentatively, and though easily followed, the reason why 0.4 is the proper ratio may not be obvious until one has computed and plotted a large number of curves of cost, when the fact must be accepted by the computer.

Example

In this case the watershed area was small, yielding an average of 30 sec.-ft., but for local reasons it was deemed best to design for a maximum of 100 sec.-ft. This meant that 100 sec.-ft. would flow only 30 per cent of the time, $C = 130$; wood-stave pipe = ⅓ cost of cast-iron, 1 cent value of horsepower-hour; $(0.30)^{0.164}$ = 82 per cent correction for part time during which 100 sec.-ft. are flowing.

Entering the diagram with Q = 100 sec.-ft., we find D'' = 81 in.; tracing down the 81-in. line to the diagonal extended through the line marked 1 cent per horsepower-hour, we read 83 in.; tracing along the 83-in. line, we arrive at the diagonal $C = 130$ with new value of D = 77 in.; tracing up the 77-in. line to the diagonal marked cost of pipe line per linear foot = ⅓ the cost of cast-iron pipe, we read on the left scale D = 91 in.; correcting for 30 per cent of time and for a 9 per cent capitalization instead of a 5 per cent, we have D = 67 in. as the value for wood-stave pipe, which is to be compared with the value of 64 in. read from a diagram prepared after Mr. Adams' method. Similarly, tracing through the diagram from the start as before at 100 sec.-ft. for steel pipe, we find D = 61 in., compared with 56.5 on the Adams diagram.

Inserting the above in the "exact formula" based on the Darcy formula we find that $D = 0.747Q^{0.481}(zy)^{0.160}$, or $D = 9.14 \times 0.747 = 6.84$ ft. Corrected for flow only 30 per cent of time gives D = 67.8 in. (Darcy hydraulic formula as basis); or based on the Hazen and Williams formula, $D = 0.786Q^{0.485}(zy)^{0.164}$, giving $D = 8.51 \times 0.786 = 6.77$ ft., which, corrected for flow of 30 per cent of whole time, gives a diameter of 65.5 in., which is to be compared with 64 in. read from the Adams diagram.

In comparing formulæ it should be noted that a difference in the per cent of capitalization is equivalent to a difference in first cost. An increase in cost per foot decreases the economical diameter. Likewise, an increase in the rate of per cent assumed decreases the economical diameter.

The determination of an approximation to the true economic diameter is readily computed mentally from the formula $D'' = 11\sqrt{x}$, when D = the diameter in inches and x = the quantity flowing in million gallons daily. This formula gives reasonably close results for values of x from 2 to 20 million gallons daily for cast-iron pipe, with $C = 100$, and a cost of 5 cents per million gallons raised 1 ft. high capitalized at 5 per cent. This formula may be corrected for other conditions according to the methods outlined.

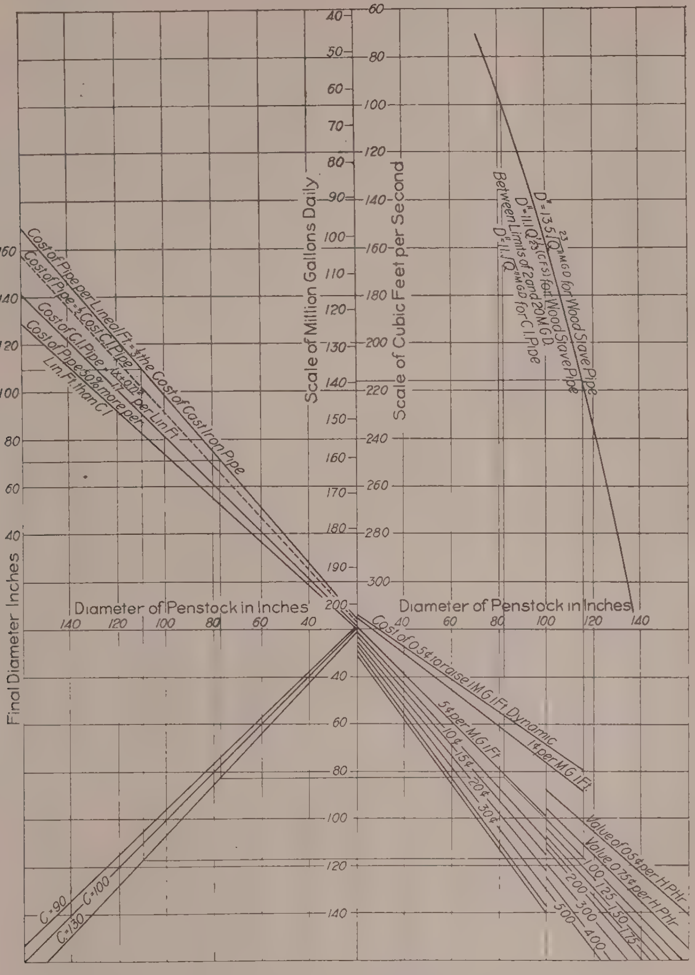


Diagram for the Economical Diameter of Water Pipes

The formula $D = 13.5\sqrt[2.3]{Q}$ was obtained by inspection of a diagram published in the Engineering Record Dec. 16, 1911, page 723. More accurately the formula reads $D = 13.5\ (e/5)^{0.138}Q^{1/2.3}$, where D is diameter required in inches; e, cost to raise 1,000,000 gal. 1 ft. dynamic; and Q, quantity in 1,000,000 gal. daily. The original formula was $2(D')^{6.87} + 1.4\ (D')^{5.87} = 8.3\ E\ b\ m\ (Q')^{2.85}$, where m is the friction factor; Q sec.-ft..; D, diameter in feet; E, capitalized cost to raise 1 sec. ft. annually on a 5 per cent basis, and b, proportional part of time water was flowing.

When the formula is used as a check on penstock computations, where wheels are installed to develop the whole flow of the stream up to a certain month, arranged in order of dryness, multiply the result obtained by the factor $(zy)^{0.1685}$ in the following table:

Month in order of dryness	Max. Q sec. ft. sq. mile	Average Q'	z	y	$(y\ z)^{0.1635}$
4	1.07	....	0.873	0.830	0.944
5	1.20	....	0.855	0.798	0.940
6	1.36	....	0.820	0.755	0.923
7	1.64	....	0.763	0.680	0.899
8	2.01	....	0.701	0.596	0.867
9	2.48	....	0.632	0.513	0.832

where z is ratio average Q' and maximum Q' and y, ratio average H_f, month by month to H_f for maximum Q'.

Effect of proportional part of whole time maximum Q flows:

Per cent of time	
10	multiply D by 0.70
20	multiply D by 0.775
30	multiply D by 0.83
40	multiply D by 0.87
50	multiply D by 0.90
60	multiply D by 0.925
70	multiply D by 0.945
80	multiply D by 0.965
90	multiply D by 0.98

The exact formula reads $D' = 0.747\ (Q')^{0.481}\ (z\ y)^{0.16}$ for twenty-four hour power, based on the Darcy formula, or $D' = 0.786\ Q^{0.466}\ (z\ y)^{0.164}$ for twenty-four-hour power, based on Hazen and Williams' formula, and $D' = 0.702\ Q^{0.485}\ (z\ y)^{0.164}$, for twelve-hour power, using Hazen and Williams' formula based on light-pressure wood-stave pipe, C = 130, value of power 1 cent per horsepower capitalized on a 7 per cent basis.

Cost of cast-iron pipe laid, class D, New England Waterworks Association's specifications at \$26.30 per ton of 2000 lb., or \$30 per long ton = $(x + 0.7)^2/1.7$ in dollars per linear foot.

Cost of Pipe Laid

Diameter, ft.	Cost pipe	Cost laid	By Formul:a
1	\$1,07	\$1.63	\$1.70
2	3.11	4.11	4.28
3	6.06	7.82	8.05
4	9.66	12.47	13.00
5	14.85	18.76	19.10

Effect of Changes in Cost of Pipe Line upon Economical Diameter

Increase in cost, per cent	Multiply by	Decrease in cost, per cent	Multiply by
50	0,935	10	1.015
40	0.945	20	1.03
30	0.955	30	1.05
20	0.965	40	1.075
10	0.985	50	1.105

An increase in the per cent of capitalization is equivalent to an increase in first cost of pipe line in its effect upon the economic diameter.

The formula is based on a factor $C = 100$—Hazen and Williams' formula, "Old Pipe."

Effect of Factor C on Economic Diameter

Average years	H'_f/H_f	C	Multiplied by
0	1.00	130	0.935
5	1.13	120	0.952
10	1.29	112	0.970
15	1.465	105	0.990
20	1.625	99	1.003
25	1.79	94	0.015
30	1.96	90	0.027

One million gallon raised 1 ft. dynamic is equivalent to 4.21 hp-hours, based on theoretical horsepower, or 5.75 hp-hours based on 73 per cent combined efficiency. The foregoing formula is based on a cost of 5 cents to raise 1,000,000 gal. 1 ft.

Instruction in the Theory and Practice of Aircraft Designs, as rigorous and exacting as any of the existing professional courses, is to be instituted at the Massachusetts Institute of Technology. The first definite step toward making adequate provision for developing the science was the detailing of Mr. Jerome C. Hunsaker, of the Corps of Naval Constructors, Department of Naval Architecture of the United States, to service at the Institute. Immediaetly after his appointment Mr. Hunsaker, who is a graduate of the school, was sent to Europe to visit the principal aeroplane and airship factories and aeronautical laboratories in England, France and Germany. On his recent return he began, in co-operation with the faculty, the arrangement of the course. It has not yet been decided whether the new science will be taught independently.

Flow of Water in Bends

Deduction of a New Formula Based on Experiments as Well as Theory and Discussion of Other Formulae

In determining upon a formula for measuring the flow of water in pipe, based on pressure differences at the concave and convex sides of an elbow (Fig. 1) Messrs Jacobs and Sooy developed the following expression for the mean velocity:

$$v = 5.60\sqrt[1.9]{H\ (r/d)}$$

where v is velocity in feet per second; H, difference in height of water columns in the gages, in feet; r, mean radius of bend, in feet; and d, the internal diameter of the pipe in feet. To this formula, established from plotting the results on logarithmic paper, Mr. N. W. Akimoff, of Philadelphia, takes exception in a paper presented Dec. 10 before the New England Waterworks Association. Mr. Akimoff does not agree either with the exponent as suggested or with the general form of the formula. The substance of the paper follows:

Omitting all reference to viscosity and assuming that an ideal fluid is dealt with, the method outlined by Prof. A. Pfarr in his book "Die Turbine" is followed. The total disposable head in the straight portion of the pipe near the bend is

$$H + V^2/2g = E, \text{ feet} \qquad (1)$$

In the assumption of parallelism and independence of separate filaments of water this will apply to each individual filament. Should the velocity of said filament be changed into v, the pressure h will be, according to Bernoulli's theorem,

$$v_2 = 2g\,(E - h) \quad \text{or} \quad v^2/g = 2\,(E - h) \qquad (2)$$

The effect of the bend will mean a new distribution of pressures and velocities, subject, of course, to the condition that the total amount of water flowing through the bend equals that in the straight portion immediately preceding it. Or, algebraically, assuming that the channel is of rectangular cross-section, the depth of which is b this condition will be

$$b\,D\,V = b\int_{r_i}^{r_o} v\,dr \qquad (3)$$

where v is a function of r, the nature of which must be established. When this is done a definite conception can be had of the static pressure (according to the radius) in each separate filament, including both the innermost and the outermost. Note that instead of a round pipe only a narrow portion of the same is concerned, comprising the innermost and the outermost filaments, as shown in Fig. 2, so that D stands for both the height of the rectangle and the diameter of the pipe.

Estimating the effect of the centrifugal force, a small element of mass, dm, of water will exert, outwardly, the pressure $dC = (v^2/r)\,dm$ upon the resisting area $b\,dl$. C is the force in pounds and v the velocity in feet per second corresponding to the arbitrarily chosen portion of the filament, of which the radius of curvature at that point is r.

But the mass $dm = (\gamma/g)\ b\,dl\,dr$, where γ is the unit weight of water, so that the actual pressure will be $dC = (\gamma/g)\ (v^2/r)\ b\,dl\,dr$. In order to obtain the value of pressure per unit of area the above will be divided by the area $b\,dl$, whence $dp = (\gamma/g)\ (v^2/r)\ dr$ in pounds per square unit, or else $(1/\gamma)\ dp = C\,dh = (v^2/g)\ (1/r)\ dr$ expressed as water column (static head) in feet or inches.

Combining with equation (2)

$$[1/(E-h)]\,\mathrm{d}h = (2/r)\,\mathrm{d}r \qquad (4)$$

Integrating between h_i and h—that is, between r_i and r

$$\log\frac{E-h_i}{E-h} = 2\log\frac{r}{r_i}$$

$$\text{so that } \frac{E-h_i}{E-h} = \left(\frac{r}{r_i}\right)^2$$

(where log means hyperbolic logarithm—that is, common log multiplied by 2.303), whence $h = E - (r_i/r)^2\,(E - h_i)$ and, by equation (2)

$$v = (r_i/r)\sqrt{2g\,(E-h_i)} \qquad (5)$$

This in equation (3) gives

$$\int_{r_i}^{r_o} v\,\mathrm{d}r = \int_{r_i}^{r_o} r_i\sqrt{2g\,(E-h_i)}\,\frac{\mathrm{d}r}{r} = DV$$

or

$$DV = r_i\sqrt{2g\,(E-h_i)}\,\log(r_o/r_i) \qquad (6)$$

If now $D = k\,r_i$ (k need not be constant for the whole bend, but is considered with reference to the section in question) this gives

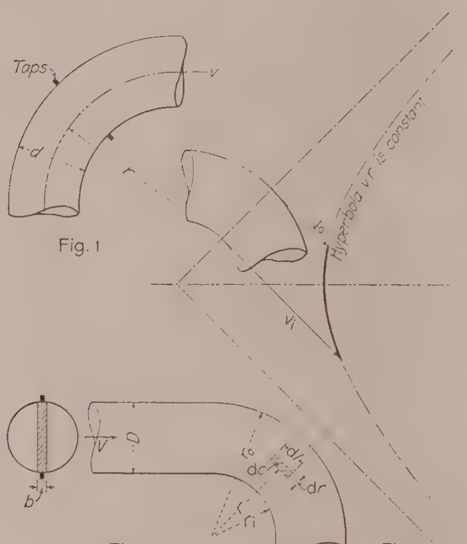


Fig. 1

Fig. 2

Fig. 3

$$r_o = r_i + D = (1+k)\,r_i \text{ or } r_o/r_i = 1 + k$$

This introduced in equation (6) gives

$$k^2 v^2/2g = (E - h_i)\log^2(1+k)$$

whence

$$h_i = E - [k^2/\log^2(1+k)]\,(v^2/2g) \qquad (7)$$

which in equation (5) gives

$$v = (r_i/r)\,[k/\log(1+k)]\,V \qquad (8)$$

so that

$$v\,r = [k/\log(1+k)]\,V\,r_i = \text{const} \qquad (9)$$

This is the equation of a rectangular hyperbola, a portion of which is shown in Fig. 3. It gives a ready way to calculate both velocities of the innermost and outermost filaments:

$$v_i = [k/\log(1+k)]\,V \qquad (10)$$

and

$$v_o = [k/(1+k)\log(1+k)]V = v_i/(1+k) \qquad (11)$$

There must, likewise, be some radius r_n at which the velocity remains unchanged so that $v_n = V$, which from equation (9) means that $r_n = [k/\log(1+k)]r_i$.

Actual computation will show that this neutral radius r_n is very nearly the same as r, the middle radius of the bend.

Incidentally a new equation may be obtained for the total flow by referring back to equation (3). Substituting v from equation (8) the total discharge is

$$bDV = b\,[k/\log(1+k)]r_i V\int_{r_i}^{r_o}(1/r)\,\mathrm{d}r$$

$$= b\,k\,r_i V = b\,D\,v_n;$$

otherwise

$$= b\,r_i\,v_i\log(1+k).$$

Since from equation (10)

$$V = v_i\,[\log(1+k)/k],$$

the above may be rewritten

$$bDV = Q = b\,r_i\,v_i\log(1+k). \qquad (12)$$

The equations (10) and (11) will enable to establish the theoretical value of the pressure difference of the two extreme filaments. Remembering equations (1) and (2) the following expressions for the outermost head h_o and the innermost head h_i can be written $h_o = E - (v_o^2/2g)$ whence, from equation (11)

$$h_o = E - (1/2g)\,[v_i^2/(1+k)^2] \qquad (13)$$

and

$$h_i = E - (1/2g)\,v_i^2 \qquad (14)$$

so that the pressure difference

$h_o - h_i = (1/2g)\,[v_i^2 - v_i^2/(1+k)^2] = (v_i^2/2g)\,[1 - 1/(1+k)^2] = (V^2/2g)\,[k^2/\log^2(1+k)]\,[1 - 1/(1+k)^2] = (V^2/2g)\,[k^2/\log^2(1+k)]\,[(2k+k^2)/(1+k)^2]$; in other words, $h_o - h_i = M\,V^2$ (15) in terms of mean velocity.

For different values of k the constant M is as follows:

$k = D/r_i$	2	1	0.66	0.5	0.4
M	0.05	0.0242	0.0167	0.0131	0.0108
N	4.47	6.42	7.72	8.73	9.61

The meaning of the constant N, forming the last line of the above table, was then given. It mostly is desirable to have the mean velocity as a function of the pressure difference $h_o - h_i$, while M is a constant for calculating the opposite, that is the pressure difference in terms of the velocity. From equation (15)

$$V = \sqrt{(h_o - h_i)/M} = N\sqrt{h_o - h_i} \qquad (16)$$

This explains the meaning of N and at the same time is the final solution of the problem.

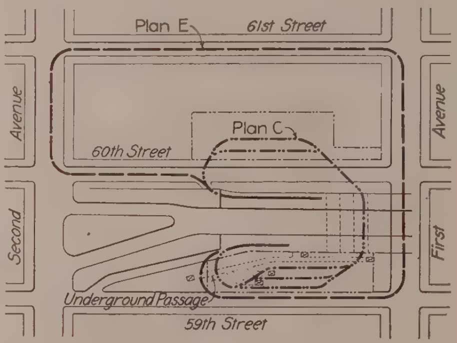
It will be seen that, theoretically, the mean velocity is proportional to the square root of the pressure difference, which, of course, was to be expected. It will likewise be seen that the constant depends upon the coefficient k—that is, upon the ratio D/r_i. This seems to be very much at variance with the assumption of Messrs. Jacobs and Sooy, who have simply assumed that the velocity is proportional to the root of the reciprocal of what has been referred to as the ratio k.

Water, of course, is not an ideal liquid and its viscosity will no doubt alter to a certain extent the formula as derived above. But there is one more important point to consider. As a matter of fact the body of water does not move inside the pipe in a "telescopic" way, as is very often assumed, its velocity increasing from nothing near the pipe to the maximum value somewhere near the center. In reality water moves in a rather complicated manner, sliding, as it were, upon a multitude of tiny roller bearings or vortex lines adjacent to the interior of the pipe. This is why the velocity is not zero in the immediate vicinity of the pipe, and not even on the pipe surface itself. Neither is it reasonable to assume that the equations (10) and (11) hold good unreservedly. In fact Professor Banki some seven or eight years ago gave a few examples where theory and facts did not quite agree in matters of this sort. This means that theory and its deductions should be used with caution.

Rapid-Transit Service on Queensboro Bridge

Modification and New Construction Required for Full Elevated, Subway and Trolley Service —Important Changes and Connections to the Bridge Necessary

TO secure proper and necessary transportation between the Boroughs of Manhattan and Queens in New York City, to properly utilize the enormously costly system of subways now under construction, and to meet the requirements of rapid-transit service fixed by the contracts entered into by New York City, the Interborough Rapid Transit Company, and the New York Municipal Railway Corporation it will be necessary in the immediate future to operate elevated railways, subways and trolley cars across the Queensboro Bridge. This service is essential to the full utilization of the extensive rapid transit construction completed or under way, will greatly benefit a large number of passengers, increase property values in Queens, and give rapid transit to a district which is without it.



Diagrams of Plans C and E

The initial expense of establishing traffic across the bridge and utilizing the latter to its fullest capacity involves the construction and location of new tracks on the bridge, changing the Queens approach for these tracks, and the construction at the Manhattan end of the bridge of a new trolley terminal to replace the present underground terminal, which must be eliminated to provide for the new subway connections. This work involves an estimated expenditure varying from $2,513,000 to $5,510,000, according to the nature of the trolley terminal provided, exclusive of the subway and elevated railroad connections in Manhattan and the new railway station at the Queens end of the bridge, which are provided for by other contracts now executed.

No rapid-transit service from Manhattan can be operated over the bridge until the changes and alterations required are completed. Rapid-transit connections to the bridge will be successively made for the Steinway tunnel, for Second Avenue elevated railway trains, and for subway trains operated by the New York Municipal Railway Corporation. This service will not interfere with vehicular or passenger traffic, and will not require any strengthening or enlargement of the bridge proper or any material changes in the main spans, which have ample capacity for the increased loads.

In a communication to the Board of Estimate and Apportionment, dated Nov. 5, Mr. Arthur J. O'Keeffe, commissioner of bridges, reports on the necessary work and estimated expense and states that it is requisite for the sum to be provided and for the board to determine the policy under which the Department of Bridges shall proceed. No portion of the bridge department construction is indefinite, except the trolley station arrangement at the Manhattan end of the bridge, the cost of which varies widely for surface and subway construction and plaza development.

The estimate is divided into four items—(1) equipment of two upper-deck tracks for elevated-railway service, $350,000; (2) relocation and reconstruction of two lower-deck tracks for subway-train service instead of vehicles and trolleys now using the floor, $1,164,000; (3) reconstruction between Marion Street and Ely Avenue of 1050 ft. at the Queens viaduct approach to adjust track grades to the new Queens station, $939,000; and (4) removal or relocation of the Manhattan subway terminal now used for Queens trolley car service at from $60,000 to $3,057,000, according to the policy adopted for the acquisition of property for a plaza at the Manhattan end of the bridge.

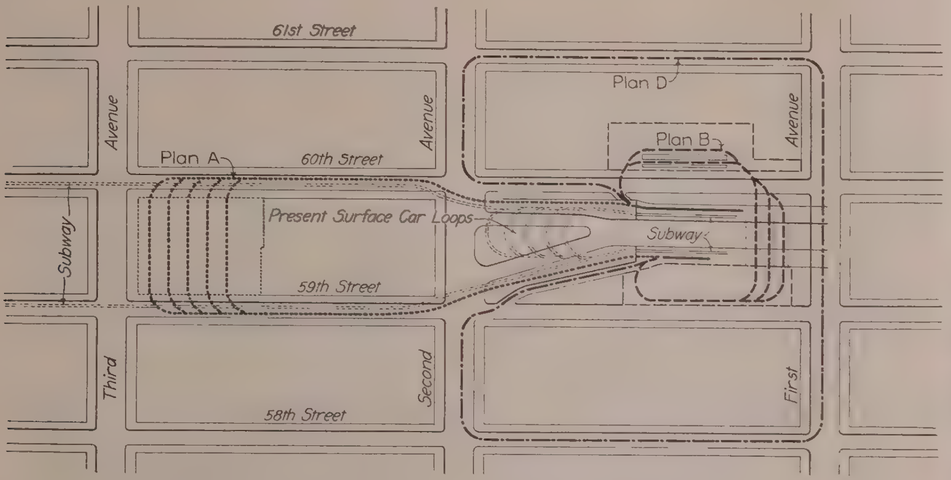


Diagram of Recommended Plan A and Alternative Plans B and D

The bridge department shows five different plans for the new trolley terminal. It recommends the adoption of plan A at an estimated cost of $1,796,000, bringing the total cost of the necessary changes and improvements up to $4,249,000. In all of these plans the space east of Second Avenue now occupied by the subway-trolley station must be taken for the connections between the lower-deck bridge tracks and the subway tracks.

Plan A provides the most direct movement of cars and the largest capacity and the most convenient arrangement of loops in a subway station on the line of the bridge axis just east of Third Avenue. The actual cost of construction is estimated at $850,000, $630,000 of which is for the underground terminal. If only about one-half of the city block between Second and Third Avenues is taken for a station and plaza the total cost of the improvement, including real estate, parking, engineering and contingencies, will be $1,796,000. If the whole block is taken, the total will amount to $3,057,000.

In plan B the cars unload on loops constructed on property to be acquired on the north side of Sixtieth Street and load on loops under the bridge arches on the west side of First Avenue. The cost of construction is $105,400, and the total cost $516,000.

Plan C is practically the same as plan B, except that the point of loading is on the north side of Fifty-ninth Street. It is objectionable because it would require the station to be on a 2½-per cent grade. The cost of construction is $145,700, and the total cost $562,000.

Plans D and E provide for the loop to be located partly on First and partly on Second Avenue. These plans are objectionable because Second Avenue is already congested. The estimated cost of plan D is $80,000 and of plan E $60,000, both sums being for construction, engineering and contingent expenses only, no provision being made for terminal buildings, the loading and unloading to be done from the street, at intermediate points of the trolley lines.

Plans have been prepared by Mr. Alexander Johnson, chief engineer, Mr. C. M. Ingersoll and Mr. A. L. Bowman, consulting engineers of the Department of Bridges.

Renewing Wood-Stave Pipe

In January, 1901, the city of Seattle, Wash., put in service a 42-in. wood-stave pipe 22 miles long, which was built the preceding year to bring the city's water supply from the Cedar River. The staves used in the pipe were 2 x 6-in. native Douglas fir, dressed on both sides, cut to radial lines on the edges and cinched together with individual steel bands. After seven years of service some staves began to show signs of failure due to decay, and since that time the pipe has required some repairs each year. As much as 500 to 1500 ft. has been entirely renewed at a time in some seasons. It is not planned to replace all the remaining original parts of the line at any one time; instead, the renewal will be made little by little, it is reported, replacing only those sections which manifest serious decay.

The rate of decay is said to vary less in different sections of the pipe as a whole than it does in the individual staves. In some cases a perfectly sound stave is found alongside one entirely decayed. The results of erosion also varied with the individual staves, the slash-sawed staves showing considerable wear at points where edge-sawed staves were practically intact. In general the quality of the timber used is thought to have had more to do with the durability of the pipe than any other cause.

Experience with this pipe, water department officials say, indicates that it would be possible to materially increase the life of such pipes by carefully selecting the timber with particular reference to soundness of fiber and conditions of sap.

Tall Concrete Chuting Tower

The substructures and towers for the viaducts of the East River Bridge Division of the New York Connecting Railroad on Long Island, Wards Island and Randalls Island involve the placing of 155,000 yd. of concrete. These have both open-pit and pneumatic-caisson foundations, with shafts up to 130 ft. high above the surface of the ground, as described in the Engineering Record of July 19, page 79.

A large proportion of the concrete is handled by the chuting system from tall hoisting-towers installed as required along the line of the viaduct and successively moved from place to place as the work advances. The towers are all of standard construction, equipped with Lakewood mixers, hoists and steel chutes, and are built at the contractor's yard with interchangeable sections assembled together to give any required height. The tallest one is believed to be higher than any previously constructed for such work. It is 5½ ft. square and 234 ft. high and has four 6 x 6-in. wooden corner posts bolted together and seated on sills resting on a grillage of four 12 x 12-in. sleepers 12 ft. long, on the leveled surface of the ground. At points 42 ft. apart vertically it is guyed by four ⅝-in. wire ropes attached to timber dead-men. It handles all of the concrete for three piers on each side, which have a maximum height of 135 ft.

Concreting Tower 230 Feet High

The best record to date is for delivering 350 cu. yd. of concrete in nine hours with a 1-yd. bucket hoisted by a double line, which will be replaced by a single line as the height increases. The adjustable discharging hopper is lifted up by the hoist rope and maintained at any height required. The 10-in. steel chutes are suspended from the 1½-in. main cables by four-part manila tackles about 30 ft. apart.

The work is under the direction of Mr. Gustav Lindenthal, chief engineer of the New York Connecting Railroad, and the tower was installed by the contractor, the Patrick Ryan Construction Company, of which Mr. H. B. Seaman is chief engineer.

Centralized Electrical Control Systems for Panama Locks

Elaborate Switchboards Graphically and Synchronously Reproducing Actual Positions of Gates, Valves and Water Levels

Electricity will effect the complete operation and control of the Panama locks. At each flight of locks a control board, which is approximately an operating miniature of the locks themselves, with indicating devices to show the positions of valves, lock gates, chains and water levels in the various lock chambers, is housed in a control house centrally located on the middle wall. In general the indications will be synchronous with the movement of the lock machinery. Complete interlocking will compel the operator to follow the proper sequence of operation.

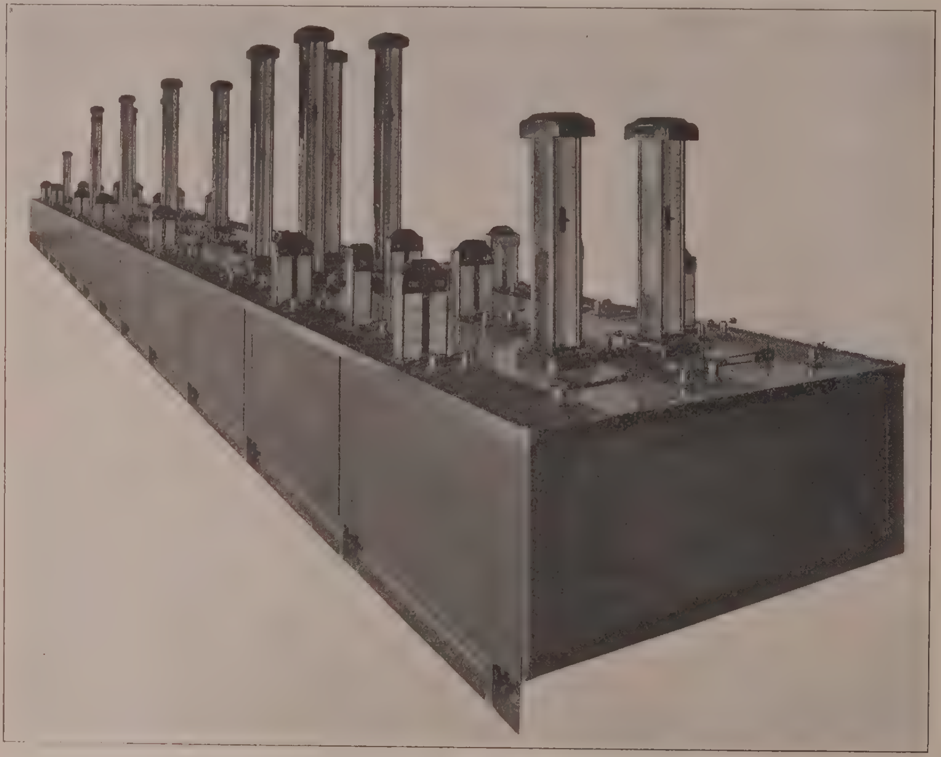
The power system, which covers not only the operation of the locks but also the towing locomotives, lights for the locks and buildings, and other machinery not directly connected with the lock control, consists of a 7500-kva 2200-volt hydroelectric power plant at Gatun Dam, a 4500-kva 2200-volt turbo-generator plant at Miraflores, a double 44,000-volt transmission line across the isthmus, four 44,000-2200-volt substations, thirty-six 2200-240-volt transmission stations for power, traction and light at Gatun, Pedro Miguel and Miraflores locks, three 2200-220-110-volt transformer stations for the central boards at the locks, and stations at Cristobal and Balboa for coal-handling plants, machine shops and drydocks.

Operation of Locks

The operating machinery for the locks is placed below the coping of the lock walls, thus affording a clear space for manœuvering ships and protecting the apparatus from the weather without erecting numerous houses. As those who have followed the progress of the Panama work know, the mitering gates consist of two massive leaves pivoted on the lock walls and operating independently of each other. Chain fenders are stretched across the canal in front of the gates, these chains being held taut when the gates are closed and lowered into slots in the lock floor when the gates are opened. The locks are filled and emptied through three main longitudinal culverts with numerous laterals; one of these culverts is in the middle wall and one in each side wall. The flow is controlled by rising stem valves so located as to permit a lock to be filled from the pool above or emptied into the pool below; the valve systems are in pairs, and the pairs are in duplicate to provide for emergencies.

As the flight of locks at Gatun is 6200 ft. long and the principal operating machines are distributed over a distance of about 4000 ft., it can readily be seen that central mechanical transmission of control would hardly be feasible, and that local control such as is generally used would mean a large operating force widely distributed and difficult of co-ordination into an efficient operating system. While the distances are less at the other points, they are still too great for effective centralized mechanical control. The situation therefore resolved itself into centralized electrical control, which reduces the number of operators, the expense and the liability to accident. For each lock a control board was constructed which permitted having all control switches located thereon mechanically interlocked so as to minimize, if not entirely prevent, human errors of manipulation.

While the control houses are so located as to afford a good view of the locks, this view is not depended upon in any way. The control board, as the illustration shows, is arranged with indicating devices which will always show the position of valves, gates, chains and water levels in the various lock chambers; and with the exception of such machinery as needs only an "open" or "closed" indication, the indication will be synchronous with the movement of the machinery.



View of Complete Electric Control Board at Miraflores Locks

The mitering gate indicator, a view of which is shown, consists of a pair of aluminum leaves shaped to conform to the top view of the gates, and swinging back into recesses as the gates do. Two of the chain-fender indicators are seen in the foreground of the view of the Miraflores switchboard. These indicators are small aluminum chains which lower into slots in the control board. A near view shows a pair of valve indicators, including the actuating mechanism. As the valves are in pairs the indicators are likewise; each indicator is similar to a small elevator. The car, a small aluminum cage behind the glass front, shows the position of the valve.

The tall cases with the graduated fronts are the water-level indicators. The specifications for these required an accuracy of 1/20 ft. To get the necessary accuracy it was found desirable to use one index within another, a coarse one giving a rough indication and a fine one showing the fractions. Lamps at top and bottom illuminate the scales.

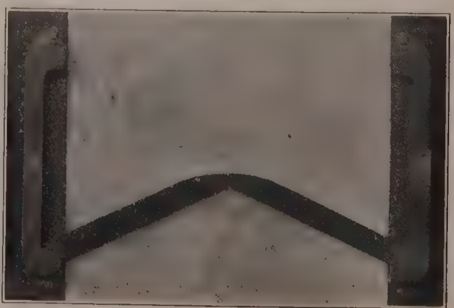
Red and green lights show whether the miter-forcing machines are open or closed.

The Miraflores control board, the one shown, is 52 ft. long; those at Gatun and Pedro Miguel are 64 and 36 ft. long respectively. Each is 32 in. high and 54 in. wide. The side and center walls of the locks, which extend the length of the board, are represented by cast-iron plates, and the water in the locks by blue Vermont marble slabs. A brass trim rail surrounds the board; removable steel plates form the sides. The control switches, which appear in the general view, operate through 90 deg. and have name plates for "open," "closed" and "off" positions.

Interlocking

The interlock system, which compels the operator to work his levers in proper sequence, depends mainly on the action of engaging bevel dogs located on horizontal and vertical bars, the movement of a horizontal bar tending to lift a vertical bar by bevels on the dogs. A horizontal bar cannot be moved without raising a vertical bar; thus if a dog on a horizontal bar rests against the upper end of a dog on a vertical bar no movement of the horizontal bar can take place, and its lever is locked.

Interlocks prevent the chain fender from being lowered until the adjacent gates have been opened, or the gates from being closed until the chain is raised. The miter-forcing machine can be operated only when the gates are closed. The valves of the side wall next to the miter gate must be closed while the miter-forcing machine is open, which means that the operator cannot create a current of water around the gates while they are open or moving. Either pair of valves may be opened first, but the interlocks are so arranged that the operator can only equalize the water between locks, and cannot allow the water to flow past one lock and flood the one below.



Index for Miter Gates

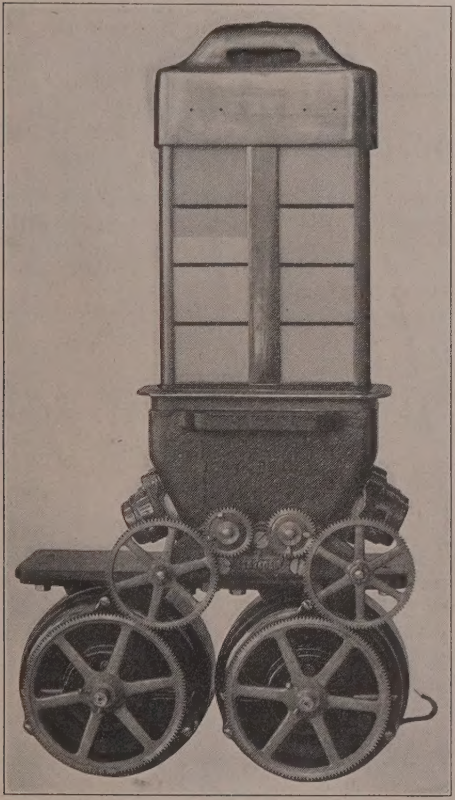
Careless cross-filling is prevented by interlocks, but these interlocks are removable by the use of a Yale lock and key to permit emergency cross-filling in dry times.

All specifications for the generating, lock-controlling and distribution system for operating the canal were prepared under the supervision of Mr. Edward Schildhauer, electrical and mechanical engineer, Isthmian Canal Commission. The electrical equipment was manufactured by the General Electric Company.

Mill-Construction Details

Floor Independent of Wall Settlement and Waterproofed at Column Points

IN a jute mill at Ludlow, Mass., special pains were taken by Mr. Charles T. Main, engineer and architect, to design a floor construction independent of the settlement of the walls, waterproofed at column points and affording recesses for electric wiring. The wall columns are made of two angles riveted together back to back, with gusset plates between projecting beyond the flanges to receive the floorbeam connections. The angles are wrapped in tar paper to prevent adhesion to the brickwork in the inclosing pilasters.



Mechanism for Panama Lock Valve Index

The I-beam girders are connected to the cast-iron column pintles, which have an I-shaped cross-section, with inside and outside sleeves respectively engaging the top and bottom cast-iron columns, with horizontal bearings on the upper and lower faces of the pintles. The tops of the wooden floor planks resting on the I-beams are flush with the upper surfaces of the pintles and are covered with a layer of waterproofing under the ⅞-in. maple boards forming the wearing surface of the floor. This waterproofing is extended over the tops of the pintles and is there covered by 16-oz. copper plates 15 in. square fitting over the sleeves that engage the inner surfaces of the upper-story columns and forming seats for the ends of the columns. The column loads are sufficient to exert a considerable pressure on the soft metal and make it act like a gasket.

In every floor five lines of wiring blocks are provided by substituting a 2¾x7-in. piece for one of the regular 4¾-in. planks forming the underfloor. This special piece rests on shoulders provided on the lower edges of the adjacent regular planks and leaves a 2x6-in. opening under it clear of the top flanges of the I-beams, on which the wires are supported.

Problems in Removing Sand from Columbia River Bar

Building Seven-Mile Jetty across Path Pounded by Pacific Waves

THE Columbia River basin includes an area of about 250,000 sq. miles, portions of which have been rapidly settled and have thus created a demand for the cheap transportation of produce and supplies. The railroad grades over the mountains naturally tend to turn the traffic to the river, and very great importance is placed upon securing a port at the mouth of the river into which deep-water vessels can safely come. Except for a narrow bar at the mouth of the river, a very satisfactory channel 40 to 60 ft. deep is always available, and during the last thirty years an expenditure of $10,000,000 has been made, exclusive of dredging costs, in an endeavor to secure and maintain an entrance to the river which shall be at all times safe for deep sea vessels.

Early Soundings

The first records of soundings on the bar were made by Admiral Vancouver in 1792. Captain Wilkes sounded over the area again in 1837, and in 1850 further careful surveys were made. These records showed great differences in the depths of water over the bar, and when the records made between 1850 and 1880 were tabulated and compared it was remarked that "the six miles of water between the two capes are a battleground of storm and water currents in which there is continual shifting and changing and in which shipping can never expect safety one month after another unless extraordinary efforts are put forth to guide the main volume of discharge."

This conclusion resulted in work being started by the Government on the south jetty in 1882, at which time it was believed that a jetty three or four miles long would provide ample directing force to enable the river current to keep the bar clear and give a minimum depth of 35 or 40 ft. By the time the jetty had attained a length of four miles a very beneficial effect was noted, and for a time a channel 30 ft. deep was maintained. The enormous quantities of sand subject to wave and current action at the bar, however, interfered with the purpose of the jetty, and the bar began to shoal slightly, advancing its outer contour seaward.

Further work on the south jetty was then decided upon and in 1902 the extension just completed was commenced. This work involved extending the jetty to a total length of seven miles, where its outer end stands in 40 ft. of water and beyond which the ocean bed dips gradually downward. The total amount of rock placed in the south jetty is about 5,783,234 tons.

Progress on the south jetty was hindered by construction difficulties; teredo action quickly weakened the trestle piling which carried the track, and the rough weather was always a menace. It is impossible for one who has not been out over the seven miles of trestle, it is claimed, to realize how forcibly the jetty is struck by the seas rolling in from the Pacific, but some idea may be obtained from the statement of the United States engineers, who aver that it was not unusual to see a rock weighing several tons carried completely over the jetty embankment. These seas do not affect the water inside the jetty, for in the shelter of its protection a deep, unchanging channel is maintained for the full seven-mile length.

Surveys made this year, however, with the south jetty practically completed, spread keen disappointment broadcast among shippers and engineers when it was known that the depth of the bar had not increased during the year since the previous survey. It is now planned to extend the north jetty and to bring its outer end to a point about 2 miles to the north of the outer end of the south jetty, thus concentrating the water discharge so as to give the outflowing current a chance to drive through the conflicting currents of the sea for a distance sufficient to keep the mouth of the channel scoured to a greater depth than at present. It is expected that this work on the north jetty will involve difficulties similar to those met with on the south jetty extension.

Dredging on the Bar.

It is estimated that 4 or 5 years will be required to complete the north jetty, but its effect should be noticeable after the first year's work, it is thought. Meantime an attempt will be made by the use of special dredging material to maintain a deeper channel than that now available. Some difference of opinion existed at first as to what effect dredging the bar would have, but the expedient of dredging was tried during the summer with results that have led to a decision to experiment on a much larger scale. The dredge Chinook worked for three months with two 20-in. suctions and increased the depth about 2 ft. It is now proposed to add two 30-in. pumps to the Chinook's equipment, and steps are being taken to provide a dredge with a much larger capacity as soon as funds can be obtained.

The construction of the south jetty has afforded a safe and dependable channel for a distance of 7 miles, and to bring the work to a satisfactory conclusion it only remains to clear a permanent channel through the small remnant of the bar at the outer end of the jetty. It has been pointed out that this presents maintenance as well as construction problems, and all concerned agree that the only method that can succeed will be one that stimulates and guides the natural forces so as to insure permanent results. The cities of Portland and Astoria have signified their willingness to stand back of any plan which promises, at a reasonable cost, a permanent channel with a minimum depth of 30 ft., and they have recently given $500,000 to help push the work to an early completion.

Both dredge and jetty work have been carried out under the direction of Maj. J. F. McIndoe, U. S. engineer.

More than 410,000 Hp can be developed from the Dettifoss waterfalls of the Jökulsa River in Iceland. The falls, which are located 37 miles from the coast, have recently been sold for $60,000 to the Nitrogen Products & Carbide Company, of England. The river, which is one of the largest on the island, has its source at the foot of the Vatna glazier and is 117 miles long.

Book Reviews

All labor laws published by Congress and by State legislatures during 1913 are outlined in a bulletin issued by the American Association for Labor Legislation. Most notable are the laws passed in five States extending the commission form of factory-law administration and those passed in eight States covering minimum wages. Several hundred labor bills were introduced in Congress alone. (New York City, American Association for Labor Legislation).

To meet the need for exact information as to the properties of slaked and unslaked lime a study of the subject has been made by the U. S. Bureau of Standards. Nineteen typical plants were visited, and the observations made were supplemented by laboratory work. In Technological Paper 16, by Warren E. Emley, assistant chemist of the Bureau of Standards, the equipments of the various plants are described and compared, with special reference to economy of operation and quality of the finished product. Some of the laboratory results are given, to throw more light on the subject. Besides descriptions of the various plants and processes considerable attention is given to the comparative heat efficiency attained by the different methods of creating the draft. The paper contains 130 pages, 30 figures and numerous tables. (Washington, D. C., U. S. Department of Commerce, Bureau of Standards.)

Mechanics for Builders. Part I. By Edward L. Bates, Lecturer at the L. C. C. School of Building, Brixton, London, and Frederick Charlesworth, Whitworth Exhibitioner, Lecturer at the Southwestern Polytechnic Institute, London. Cloth, 5 x 7½ in.; 201 pages; 172 illustrations. New York, Longmans, Green & Company. $1 net.

The book is intended to present only those principles of applied mechanics which are essential to the student of building construction who desires to follow intelligently his class of work and to understand how to design and carry out any work with which he may be intrusted. It is notably elementary and consists of two parts, designed respectively for first and second-year courses. It gives explanations, illustrations, analyses and simple diagrams and formulæ, with problems and answers, on force, stress and strains, moments, framed structures, girders and simple machines. It discusses friction, stability of masonry structures, work and power, and has convenient tables of logarithms and antilogarithms to four places. Excepting a few simple trigonometrical functions, the mathematics required are limited to arithmetic.

Pocket Companion for Engineers, Architects and Builders. Flexible leather, 5 x 7 in.; 400 pages: numerous illustrations and tables. Pittsburgh, Carnegie Steel Company.

This handbook, although in the general line of section books published by the large rolling mills, is more extensive and complete than such publications usually are, and contains a large amount of useful information conveniently arranged, much of it not elsewhere available, which makes it a valuable auxiliary for an engineer's pocketbook and a necessity for the bridge and structural designer.

The general scope of the book embraces three principal divisions, namely, mill material, elements of sections and design tables, and general information for designers. The first section, covering 132 pages, gives diagrams and tables of shapes, plates, bars, special sections, pipes, bolts, screw rods, eyebars, loop eyes, turnbuckles and rivets manufactured by the Carnegie Steel Company. In order to further the standardization of quantitative specifications there are also included the complete text of the 1909 specifications of the American Society for Testing Materials, for structural steel for bridges, structural steel for buildings, structural steel for ships, structural nickel steel adopted 1912, boiler and firebox steel, 1912; boiler rivet steel, 1912, and steel reinforcing bars, 1912. There are, besides, specifications for design and workmanship of fabricated structures.

The division of elements of sections covers 44 pages and gives definitions, formulæ and totals of values for the radius of gyration, moment of inertia and section modulus of standard sections, geometrical figures and simple riveted members.

General information covers 234 pages and discusses beams, plate girders and columns, their loading and stresses, and gives formulæ and examples of their use, with extensive tables of safe loads. Data of standard detailing for structural steel members, including riveting and rivet values and pin bearings, are given. Floor and roof construction and loads, flat and segmental arch-buckle plate and concrete floors are also considered. There are miscellaneous tables of the strength of timber, qualities of materials, equivalents of measure, metric conversion, mensuration and convenient mathematical tables.

Among the important new data not presented in the former editions are the supplementary light-weight I-beams and H-beams, and several pages are devoted to the new product for concrete reinforcement and for steel sheet piles.

A positive improvement has been made by the use of 1000-lb. units instead of the former 2000-lb. (1 ton) units in loads and stress tables, thereby simplifying the computations by the elimination of division and requiring only a proper pointing off of decimals.

The book is handsomely printed on India paper so thin that the 400 pages occupy a thickness of only ½ in. It is intended for free distribution to designers and users of structural steel on application at the nearest district office of the company, and also for sale to engineering and architectural students.

Letters to the Editor

Ozone as a Room Disinfectant and Deodorizer

Sir: In the Engineering Record of Dec. 6, page 645, there is an abstract of two articles on ozone which recently appeared in the "Journal" of the American Medical Association and a comment by Dr. Steinmetz which appeared in the "Electrical World." Without attempting to defend the former articles I should like to make a few suggestions in regard to the comment.

It is certainly true that in some cases the concentrations of ozone used for experimental purposes have been far in excess of that which could possibly be used for purifying air; but this criticism cannot be applied to much of the work recorded in the above-cited articles. Dr. Steinmetz quotes the articles as saying that "ozone in higher concentration destroys odors." I fail to find this statement. The nearest approach to it is well modified, and an examination of the experimental results does not lead to any such conclusion. Furthermore, I do not find it difficult to believe in the possibility that ozone in high concentration may act differently from ozone in low concentration. Other substances certainly act in this way—nitric acid, for example.

In regard to the statements concerning the resistance of dry bacteria to ozone, it must be remembered that it is exactly bacteria in this dry state which occur in air, and any substance to be used effectively as an air disinfectant must be capable either of supplying moisture or of acting on comparatively dry bacteria. The case of water sterilization by ozone is not at all comparable to that of air purification. The statement, "as the action of ozone is oxidation or combustion, bacteria contained in jelly could be affected only by the complete destruction of the jelly," is somewhat startling to one who has worked with bacteria and disinfectants. Chlorine is also a disinfectant and an oxidizing agent, and while it is perfectly true that the chlorine will act on organic matter as well as on bacteria when both are present, it is certainly not true that all this organic matter must be destroyed before any action on the bacteria can take place. If this were true, sterilization of sewage by chlorine would be out of the question as a practical method.

After all, is not this discussion of the action of small amounts of ozone on bacteria quite irrelevant to the real question at issue? Aside from proving or disproving advertising claims, has it any real value? It seems to be the general opinion that there is little danger from infection by bacteria contained in the air of a schoolroom or office. It is necessary to have much closer contact than is usually obtained under such conditions. It seems to me that from the standpoint of "good air" there are two questions to be settled. First, is ozone an effective agent in destroying the odors given off from the body? Second, if so, is ozone so used injurious to health? Has there not been sufficient work done to answer these two questions? John F. Norton.

Laboratory of Sanitary Chemistry,
Massachusetts Institute of Technology.

Dragging Down the Profession

Sir: In an editorial in your issue of Dec. 13 you decry the affiliation of a number of engineers with the American Federation of Labor in an effort to raise standards of pay. No engineer worthy of the name desires to drag down the profession, but with not a few in the minor positions it is a question of bread and butter. They can reasonably afford to spend a few years at scant pay, but they are as prone to settling down and marrying as any other class of men, and they find themselves unable to sustain their families on the pap fed them—valuable experience and the honor of the profession.

Take the railroad field, for instance. The technical graduate starts at a figure considerably below that obtained by the young brakeman. The company owns him, sending him anywhere, in any kind of weather, on a minute's notice. If he receives a telegram at 11 p. m. to take the midnight train to Springville, he must go. If he has raised enough money to take his girl to the theater and has made all arrangements, he may get orders at 4.59 to work all night on a report, so the boss can go on a fishing trip Saturday. He is the scum of the earth and his personal affairs are never considered.

A little of this will not hurt him, but what is the ultimate reward? On some roads it may be a notice, after ten years' service, when he may or may not have attained the princely salary of $125 per month, that they will henceforth be able to get along without him. Many of the railroad managements have never discovered that rainy days and sunny days keep coming in succession in the same old way; consequently when the November earnings make a bad showing they think they are never going to have any more money to spend and they throw away their organizations, starting anew the following spring, when the flurry has blown over.

One railroad has a reputation for protecting its men in hard times, but consider how highly it rates them at all times. It publishes a leaflet announcing promotions, with sketches of the careers of those promoted. I have one now that a friend sent me. Half a dozen men have been promoted to positions as supervisors. One of them is only 29 years old, but he has been with the road twelve years; one has seen only ten years of service. The six are from 29 to 36 years old, and have been with the company from ten to sixteen years.

The supervisor does not sleep on a bed of roses. He goes up and down the road at all hours of day or night. He has to leave his Sunday dinner to clear up a wreck, and if in the dead of night a sleet storm should freeze up his switches he is compelled to turn out to get the line open. Having worked all night he spends the next day making reports and answering telegrams from the chief clerk. He has little time to remember that he has a profession, less to read of the efforts of his fellow-engineers and none at all to attend meetings. On some railroads his pay is $100 per month, on some $125, on some perhaps much more.

The supervisor is a loyal soul, and federations of engineers will not interest him. His strenuous life has driven away his outside interests, the railroad has become his god, and he takes it as a matter of course that he is called upon to work twenty-five hours a day. It is the younger man, who has become less estranged from the outside world, who sees his classmates signing themselves assistant chief engineer or business manager and living in comfort while he is still an obscure rodman, who looks at the supervisor or the division engineer, sees how hard he works and wonders if the goal is worth striving for—he it is who perceives that he is virtually excluded from the profession, and that if the unions are to grab all the loose change that the railroad has, self-preservation demands that he take steps to get his share.

New York City TRANSITMAN.

Viaduct Floor of Hollow Concrete Tile

Sir: As a conclusion to the article published in the Engineering Record of Dec. 6, page 647, it may be of interest to know that the Trent Road viaduct is now completed. Before opening the bridge to traffic we subjected it to a test load by running over it a 16-ton road roller. As stated in the article, the bridge was originally designed for a 15-ton road roller. The roller was placed in the center of the longest slab, having a span of 22 ft. 3 in. from center to center and a depth of 16 in. Under this loading the slab deflected only 1/40 in., as measured by an accurate device for reading deflections. This is less than 1/10,000 of the span, and was more than satisfactory to us, indicating the exceptional strength of the concrete tile construction.

The reduction of weight of the viaduct accomplished by the use of the hollow tile was 140 lb. per square foot over the solid slab design originally contemplated, or a total reduction of 280 tons for the entire viaduct. The construction gave the best of satisfaction, both in the matter of speed of erection and simplicity of centering, and also enabled all reinforcement to be placed, including top bars, before the concrete was poured.

The viaduct has met the approval of the county and railroad authorities, and from an æsthetic standpoint is a much more pleasing structure in appearance than could be made from either steel or wood.

J. W. STRACK,
Spokane, Wash. County Engineer.

Surge Tank at San Francisquito Power Station No. 1

Sir: Replying to the query of Mr. R. D. Johnson in the issue of the Engineering Record of Oct. 11, page 420, regarding the surge chamber for San Francisquito Power Station No. 1, the following points may be noted:

The numerical data used by Mr. Johnson are, with the exception of the total static head H, all in error. Using his notation, the following corrections may be noted: L, 41000 instead of 39600; A, 100 instead of 81; C, 0.76 instead of 0.835; V_1, 6 instead of 7.4; V_2, 10 instead of 12.34.

Thoma's formula, as printed, is typographically wrong in the radical of the value of F. This should be corrected to read $\sqrt{Z}(Z + 2)$. The incorrect numerical values used all tend in the same direction and give a result much too small. Making the correction in the formula and substituting the above correct numerical values, the results for a cylindrical standpipe are $F = 2490$ sq. ft., instead of 1150, and diameter = 56.3 ft., instead of 39.

Regarding the choice of a plain standpipe rather than the differential regulator referred to by Mr. Johnson, it must suffice to say that the matter was considered with care by the city's engineers in the light of the best information attainable at that time (1910), and for reasons which seemed valid it was decided to install a plain chamber, conical or tapering for the portion excavated below the ground level and cylindrical for the part above ground.

The condition of dead-beat movement for demanded load on a 40 per cent change of load was only one of a great variety of conditions imposed, nor was it the most severe.

The actual flow may be anything from that corresponding to a velocity of 10 ft. per second downward, and under a static head which may vary due to change of reservoir level through some 75 ft. So far as surge chamber design is concerned, the conditions may, therefore, be stated as follows:

Given, a reservoir level anywhere in its 75 ft. of range with a resultant possible upper conduit velocity varying from 10 ft. per second for the higher levels down to 7 ft. per second for the lowest level, and with actual values of the velocity anywhere within these upper limits according to the load demand, and with a possible sudden change varying from as much as 100 per cent of the lower rates of flow to perhaps 60 per cent of the higher rates. Required, a surge chamber which, under all conditions, will provide substantially dead-beat movement of the water level, at least under demanded load.

The actual design was based upon the careful solution by a step-by-step integration process of some twenty or more different possible combinations of conditions. The fundamental equations lying at the basis of this solution are those well known for the surge chamber and regarding which there is no question among engineers. They are precisely the same equations from which have been derived various approximate algebraic solutions and formulæ. It cannot be expected or maintained, however, that the results given by such approximate algebraic formulæ will be as accurate as those derived by the longer but more accurate stepwise integration process.

The results of these careful and extended numerical integration processes indicated that no chamber essentially smaller than that adopted could be expected to meet the conditions imposed.

The particular reason for dead-beat movement of the water was in order to provide as favorable conditions as possible for governing. This particular plant is to be the governing station for a total system aggregating perhaps 150,000 hp, and possibly without steam stations, with heavy steam turbine rotors to float on the line. Under these conditions the very highest and most exacting governing conditions are naturally required for this station, and in these facts the city's engineers have found their justification for recommending and adopting, as far as practicable, a surge chamber which will insure dead-beat movement of the water level under any probable change in the way of demanded load.

The conditions for dead-beat movement under rejected load are much more exacting, and numerical cases worked out in detail show that there will be, even with the size of chamber adopted, some slight surging under rejected load.

These general conclusions have all been verified by experimental investigations by means of a model installation using the law of kinematic similitude as developed and applied to the surge chamber problem ("Transactions" of the American Society of Mechanical Engineers, Vol. 34, page 359).

W. F. DURAND,
Member Board of Consulting Engineers,
Bureau of Los Angeles Aqueduct Power.
Leland Stanford University, California.

[A copy of Professor Durand's letter was submitted to Mr. R. D. Johnson for a reply, which follows.]

Sir: Referring to Professor Durand's reply to my query regarding the surge chamber for the San Franciscquito power station, I am not surprised that the data which I interpreted from the previous article were all in error, as he states. I anticipated just such a disclosure. My opinion, however, that a 30-ft. cylindrical regulator is more than sufficient to produce dead-beat conditions for a 40 per cent velocity change is now only slightly modified. It is true that this capacity would not produce substantially dead-beat conditions for more than about 15 per cent velocity changes when applied to the correct requirements as now stated, but the amount of surging even for a 40 per cent velocity change would be a negligible quantity, probably not more than 5 or 6 ft., or, say, three-quarters of 1 per cent of the head. This tank would need to be carried about 50 ft. above high forebay level to prevent spilling over in case of a sudden interruption when operating at full load. The justification for the excessively large size of this San Francisquito chamber seems to be due to the omission of differential action combined with three unusual conditions, viz.:

First, the static level within the chamber is subject to a variation of 75 ft. with no velocity, due to fluctuation in the pond level.

Second, the shape of the chamber has been made conical.

Third, it is required to produce substantially non-oscillatory conditions at low water for an increment of load represented by a sudden change in the penstock velocity of 7 ft.

If the pond level has dropped 75 ft. the water surface at no load is already well down in the cone, where the area is nearly cut in two, and it is evident that a cylindrical shape of a volume equivalent to that of the cone is better adapted to this condition. It is ordinarily unnecessary to provide greater capacity in a surge tank than is sufficient to take care of normal or, perhaps, somewhat abnormal load increments for which it is intended to try to maintain steady frequency. Large increments of load due to short circuits or following interruptions usually involve so many conditions which interfere with good service that they are classed as accidents, the evil consequence of which cannot be much improved by larger surge-tank capacity. It is for this reason that I am surprised at the severity of the requirement which has governed the design of the San Francisquito surge tank. The sudden demand for 700 sec.-ft. under a head of 800 or 900 ft. represents an enormous load change for which to expect anything like uniform frequency; but granting the necessity or desirability of maintaining "substantially deadbeat movement of the water" for this condition when the tank is already depleted 75 ft., then I may say that the large capacity of this conical reservoir is not only justified but not even adequate.

R. D. Johnson,
Niagara Falls, N. Y. Hydraulic Engineer.

To Find the Point of Intersection of Two Grades and Vertical Curves

Sir: A letter entitled "To Find the Point of Intersection of Two Grades" appeared in the Engineering Record of July 19. The writer's method of solving this problem is a little shorter and more easily remembered.

In the accompanying figure a and b are the elevations of any two known points on g_1 and g_2 respectively. These points are d 100 ft. apart, and the grades are algebraically expressed in feet per 100. Obviously h, the elevation of g_1 over the point b is equal to $a + g_1d$. The two grades approach each other by g_1g_2 feet for each 100 ft. that the point of intersection is approached. The horizontal distance from b to the point of intersection is, therefore, $100(h - b)/(g_1 - g_2)$, in which expression h, b, g_1 and g_2 are expressed in feet and the grades expressed algebraically.

The following formulæ state the relations of the tangents, external distances, and vertical grade deflections which it is generally desirable to know in solving the bulk of the vertical curve problems. The two grades, G_1 and G_2, and the elevation of their intersection point, V, must always be given. The parabola is then determined either by the choice of a particular length of "tangent," T, external distance, E, or vertical grade deflection per station, J.

First Case.—T, G_1, G_2 and V are given. Find E and J.

Figuring the grades from V, the distance, $DB = T\ (G_1 - G_2)$. It follows from the law of the parabola that

$$E = (T/4)\ (G_1 - G_2) \qquad (1)$$

Again from the law of the parabola $DB = (J/2)\ (2T)^2 = 2T^2J$, then $2T^2J = T\ (G_1 - G_2)$, and

$$J = (G_1 - G_2)\ (2T) \qquad (2)$$

Second Case.— E. G_1, G_2, and V are given. Find T and J.

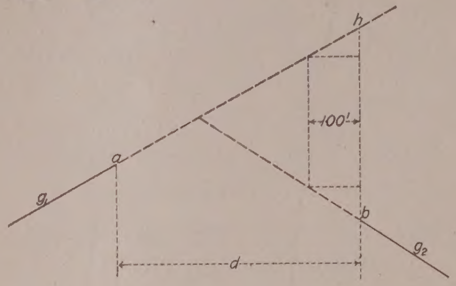
It follows directly from equation (1) that

$$T = (4E)/(G_1 - G_2) \qquad (3)$$

By substituting this value for T in equation (2), it becomes

$$J = (G_1 - G_2)^2/(8E) \qquad (4)$$

Third Case.—J, G_1, G_2, and V are given. Find T and E.



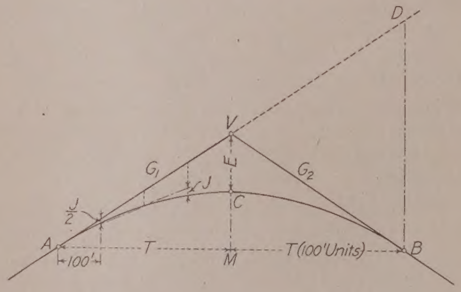
These values follow directly from equations (2) and (4), respectively.

$$T = (G_1 - G_2)/(2J) \qquad (5)$$

and

$$E = (G_1 - G_2)^2/(8J) \qquad (6)$$

When a train passes in either direction over a hump in the track, whether the hump is on the summit or not, the front end will speed up and tighten the line of traction; but whenever a train swings across a sag the slackening line of traction causes the cars to slat each other back and forth and to crowd the rails, while the thrust of the changing direction causes a heavy pounding of the track. For these reasons good practice recommends parabolas of $J = 0.05$ for sags and $J = 0.10$ for humps. $J = 0.50$ is the maximum limit for railroad work. Prof. C. Frank Allen's field book gives a good exposition of the methods for figuring vertical curves in detail. It has been pointed out to the writer that the slide-rule can be used very handily in these computations. Set T on the C scale under E on the A scale. For any intermediate point, find the vertical offset from the grade line to the curve on the A scale over the distance of the point from the beginning of the curve on the C scale.



Kansas City, Mo. Robert S. Beard.

Calculation of Dock and Bulkhead Walls

Sir: Theoretical computations based on the commonly accepted theories for earth pressures cannot be expected to give reliable results, and they are, therefore, of little practical value. The explanation of this lies in the assumptions governing the earth-pressure theories, and so far very little has been done to correct them. From time to time someone appears before an engineering society and reads a paper referring to these subjects and a sudden interest is awakened, but the next day those who were present at the meeting use the old theories over again. In the paper on page 325 of the Sept. 20 issue of the Engineering Record the author proposes as a factor of safety an increase of 25 per cent in all active earth pressures and a decrease of 25 per cent in all passive pressures. This would be conservative if the actual values of these pressures were known. But as no cohesion in the earth has been considered in the formulæ, these precautions are unnecessary, as the factor of safety involved, although unknown in amount, is far larger than the safety factor proposed. Several well-known European experts have tried to establish for earth-pressure formulæ coefficients providing for cohesion of the different kinds of earth. Prof. E. Jacoby, of Riga, Russia, has even gone so far as to propose the use of the formulæ without any increase for safety, stating that the cohesion, not considered, has added a safety factor of at least two. Dr. H. Ehlers, of Hamburg, who has made experiments on the structures in the harbor of that city, also states that the active earth pressures are far less and the passive pressures much greater than the calculated values. An average of ten experiments shows that the calculated pressures are 2.41 larger than the actual; this confirms the statements of Professor Jacoby. Of course, this ratio will vary for different kinds of earth and must be determined experimentally in each case.

The dimensioning of bulkheads and sheet pilings has also received attention abroad, and Professor Jacoby has arrived at the conclusion that a factor of safety of two is sufficient. The permissible maximum unit stresses for spruce and white pine would, therefore, be about 1500 lb. per square inch; for oak, 2000; and for yellow pine, 2500. Professor Jacoby is supported by statements in Prof. Max Möller's book, "Grundriss des Wasserbaues," Part 1, page 108, published in 1906. The active and passive earth pressures noted in the paper by Professor Fellenius ought to be respectively lessened or increased by from 50 to 100 per cent. Should the earth pressures be calculated and used without any consideration as to cohesion the 25 per cent safety factor could be dropped altogether.

New York City. A. M. Case.

A Serious Limitation upon Economy in the administration of the appropriations made for the United States Geological Survey, according to the annual report of the director, recently made to Secretary Lane, results from the discordance between the fiscal year and the field season. The necessity of making field plans and starting field projects on July 1, when the natural season for field work is well advanced, involves not only great inconvenience but serious loss. Congress has in some measure relieved this unnatural situation by making certain of the appropriations for the field service wholly or in part immediately available. Yet even this plan fails to afford practical relief when, as last year, the appropriation act was not finally passed until late in the summer (Aug. 24, 1912). It is difficult to estimate closely the loss involved, but it can be safely stated that, as a result of this delay, probably never before has the unit cost of field work in the operations of the Geological Survey been so high as this last year.